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Application of the Storm Water Management Model

Volume I

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Application of the Storm Water Management Model

Volume I

by

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This project focuses on methods of maximizing the use of water resources in a planned urban environment, while minimizing their degradation. Particular attention is being directed towards determining the biological, chemical, hydrological, and physical characteristics of storm water runoff and its corresponding role in the urban water cycle.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

PREFACE

The overall goal of this research was to evaluate the water resource plan for The Woodlands, Texas, and to make recommendations, as necessary, to maximize its effective utilization through alterations in design and management. Any recommended alterations were to be critically evaluated as to their compatibility with the natural environment.

Collection and utilization of stormwater runoff for recreational and aesthetic purposes was a major feature of the water resources plan at The Woodlands. Control of downstream flooding was also of great importance and so storage reservoirs, in the form of recreational lakes and wet weather ponds, were created by the developers. Water quality was a concern if the impoundments were to be aesthetically appealing and/or suitable for recreation. Therefore, a major sampling and analytical program was designed to monitor water quality and quantity at different locations in the developing area. The Storm Water Management Model (SWMM) provided the focal point for combining the water quality and quantity data into a predictive tool for design and management purposes.

SWMM was originally developed for highly urbanized areas and, therefore, was calibrated for this project in an urban watershed (Hunting Bayou). Subsequently, SWMM was modified to model runoff and water quality from natural drainage areas, such as The Woodlands. Because of the lag in the construction schedule at The Woodlands, the dense urban areas were not completed during the project period. Consequently, Hunting Bayou and other urban watersheds were sampled to provide a basis for predicting pollutant loads at The Woodlands in the fully developed state.

Water analyses included many traditional physical, chemical and biological parameters used in water quality surveys. Pathogenic bacteria were also enumerated since the role of traditional bacterial indicators in stormwater runoff was not clear. Algal bioassay tests on stormwater were conducted to assess the eutrophication potential that would exist in the stormwater impoundments. The source, transport and fate of chlorinated hydrocarbons in stormwater runoff was also investigated.

Several of the large Woodlands impoundments will recieve reclaimed wastewater as the major input during dry weather. Besides their use as a source of irrigation water, the lakes will be used for non-contact recreation -- primarily fishing and boating. Because the reclaimed wastewater must be disinfected, there was a concern about disinfectant toxicity to the aquatic life in the lakes. Consequently, comparative fish toxicity tests were conducted with ozone and chlorine, the two alternatives available at the water reclamation plant.

Porous pavement was considered by the developers as a method for reducing excessive runoff due to urbanization and an experimental parking lot was constructed. Hydraulic data was collected and used to develop a model compatible with SWMM, to predict the effects of using porous pavement in development. Water quality changes due to infiltration through the paving were also determined.

Hopefully, the results of this project will contribute in a positive way to the development of techniques to utilize our urban water resources in a manner more compatible with our cherished natural environment.

ABSTRACT

Stormwater runoff from urban areas has been recognized as one of the major contributors to pollutant loadings in natural rivers, lakes and estuaries. To evaluate these loadings, characteristics of stormwater runoff from an urbanized area and an undeveloped site were quantified for several key water quality parameters at selected sites.

The Storm Water Management Model (SWMM) was modified to allow for modeling of 1) separate sewer systems, 2) effects of urbanization on baseflows, 3) performance efficiency of natural drainage systems, 4) cost efficiency of natural drainage systems, 5) four more water quality parameters - COD, Kjeldahl nitrogen, nitrates and phosphates, 6) hydrologic effects of porous pavement areas. A new subroutine was programmed for each of these objectives and included in the SWMM. All new subroutines are user options.

The resulting SWMM version can model storm periods separated by zero or low rainfall, eliminate all dry weather flow resulting from sewage, compute baseflow recessions, model flow in natural nonuniform cross-sectional channels, determine costs of natural drainage systems, model eight user selected water quality parameters for as many as 20 land uses, and evaluate the performance of porous pavement.

The SWMM was applied to the urbanized Hunting Bayou watershed in Houston, Texas, and Panther Branch, an undeveloped watershed where the new community of The Woodlands is being developed in its downstream reaches.

Because the original water quality predictions from SWMM verification and calibration runs were too low for urbanizing areas, a user option to input the actual loading rate and removal factor for each pollutant under consideration was introduced into the SWMM. This approach was verified by comparison to observed data on Panther Branch and Hunting Bayou and then applied to Swale 8, one of the tributary areas to Panther Branch, which is currently being urbanized. A management strategy to control storm water in a manner compatible with the natural environment and proposed natural drainage network was developed for Swale 8.

This report was submitted in partial fulfillment of Grant No. 802433 by Espey, Huston and Associates through Rice University under the sponsorship of the U. S. Environmental Protection Agency. This report covers the period from September 1, 1973, to September 1, 1976, and work completed as of September 1, 1976.

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SECTION 1

INTRODUCTION

Numerous studies and ongoing data collection programs have provided a comprehensive understanding of the changes in the hydrologic characteristics of a watershed during and after urbanization. Both quantity and quality of runoff are affected. Increases in peak discharge rate and total volume of flow are well documented, but the reduction in quality of urban stormwater has only recently been recognized as a major problem. Consequently, flooding from increased runoff and water quality degradation are two major problems in urban hydrology.

The development of impervious areas such as roofs, streets and parking lots in urban areas results in a severe limitation of the infiltration capacity of an urban watershed. Stormwater management has generally consisted of collecting all the runoff in gutters and discharging it into a conveyance system of storm sewers and channels which are tributary to a nearby stream, lake or ocean. An efficient urban drainage system has generally implied the use of storm sewers and lined and rectified ditches. Although local flooding problems were solved by this system, increased time of concentration and higher peak flows which are generated tend to create severe flood problems downstream. Also, impervious areas were found to have very few urban pollutant assimilative properties. In fact, impervious areas tend to generate urban pollution that is not amenable to street sweeping and therefore, much more difficult to control (1). The increase in flow velocities in improved channels creates a high erosion and scour potential, thus aggravating pollution problems in receiving waters.

An alternative drainage scheme termed natural drainage is now being considered. The natural drainage concept is based on the premise that typically narrow and deep drainage ditches and storm sewers are undesirable. Therefore, existing drainage channels are utilized to the fullest extent possible and any new channels are constructed and lined with native vegetation to function similarly to the existing channels. In order not to exceed the capacity of natural drainage channels, runoff rates must be approximately the same before and after urbanization or the runoff rates must be reduced to runoff rates at natural watershed conditions.

The major method to reduce runoff rates is the use of minor detention and retention ponds. In areas where a number of contiguous impervious surfaces exist or are planned, the use of porous or pervious pavements becomes a viable alternative. Porous pavements and minor ponds provide design storage so that they may be used to reduce runoff to preurbanization levels, but, more importantly, they can be used to capture the initial runoff or "first flush" volume which most studies indicate to be the most degraded in terms of pollutant concentrations. All of these runoff controls are utilized at The Woodlands, a new community being developed near Houston, Texas.

In 1973, the U. S. Environmental Protection Agency (EPA) sponsored this study to quantify the effects of the runoff controls discussed above. The Storm Water Management Model (SWMM), originally developed under EPA sponsorship for cities with conventional drainage systems and combined sewerage, was selected for this purpose because of its comprehensive modeling capabilities. The present study was undertaken to modify the SWMM in order to evaluate the effects of runoff controls as used in natural drainage systems as practiced in surface drainage design at The Woodlands and in Houston, Texas.

The SWMM was modified through inclusion of several new subroutines and debugging of existing subroutines. The resultant model was then applied to Swale 8, the major watershed area at The Woodlands where urbanization is progressing under natural drainage concepts.

This final report summarizes the three years of research and development effort expended in achieving the study objectives.

SECTION 2

CONCLUSIONS

The Storm Water Management Model (SWMM) released February 1975, referred to in this report as the original SWMM version was extensively modified by this project. The capabilities of the modified version have been expanded to model runoff and water quality from natural drainage areas. The study areas where the new capabilities were tested are The Woodlands and Houston, Texas. During the course of this study the following conclusions were reached:

1. After correction of errors in infiltration rate computation, the modified SWMM prediction of observed peak and total discharge were relatively accurate but the pollutant concentration predictions, which are dependent on exact hydrograph replication, could not be modeled for the study area.

2. Although runoff from natural drainage areas is of better quality than that from areas with conventional storm sewer drainage, the effect of construction activity in both types of areas could not be determined by the original SWMM version. The predicted values were always too low. But, modeling of erosion from construction activities is now possible by use of the modified SWMM version.

3. Laboratory methods used to determine biochemical oxygen demand data produced inconsistent results and therefore the biochemical oxygen demand modeling proved unsatisfactory. Data for chemical oxygen demand were more consistent and subsequently used to model chemical oxygen demand during this study.

4. It was determined that the functional relationship between pollutant mass and runoff volume could be linearized by the use of logarithmic transforms. The resultant linear equations can be used to determine loading rates and total pollutant transport from a watershed area.

5. The exponential pollutant removal or decay coefficient can be considered as a constant in all geographical areas. The modified SWMM allows for selection of the value of this coefficient by the user.

6. Water quality modeling capabilities of the SWMM have been considerably improved. The modified SWMM can reasonably predict mass flow rates (and pollutographs, if the hydraulic modeling results are accurate) for suspended solids, chemical oxygen deamand, nitrates, phosphates, or any other pollutant for which loading rates may be determined. These capabilities were proved by modeling observed events at the study areas where even transient land use such as construction activities were successively modeled.

7. The modified SWMM can be used to model runoff events generated by distinct periods of rainfall separated by periods with zero rainfall. The original SWMM did not have this capability.

8. The modified SWMM can reflect interaction between surface water drainage and groundwater conditions by determination of recession flow rates from input recession characteristics.

9. The modified SWMM can transport flow through natural channels with a minimum of input data requirements. Each natural channel is described by a series of coordinates and the program now calculates the area-discharge curves which formerly had to be input for each natural channel.

10. The modified SWMM can determine the cost efficiencies in the use of natural drainage systems relative to those for conventional drainage systems using either user supplied or default unit cost estimates.

11. The modified SWMM can provide a detailed analysis of storage and flow into and out of porous (pervious) pavement systems. The drain outflow, surface runoff, and storage volumes can be determined; but the lack of comprehensive data precluded the modeling of water quality in porous pavements.

12. The modeling schemes developed during this study require considerable input data preparation and consequently the modified SWMM, when applied to natural drainage systems, is more user dependent.

SECTION 3

RECOMMENDATIONS

The water quality relationships developed in this study should be verified with data from other watersheds across the country to determine the possibility of data transfer. New or revised coefficients may be developed for nonhomogeneous watersheds or different climatological regions. If linearization of the log transforms of water quality data is found to be universally applicable, the prediction of water quality will be facilitated.

More data from Panther Branch and Hunting Bayou watersheds are necessary to determine if the empirically derived relationships used by the modified SWMM are generally applicable and also to improve the accuracy of those relationships. Additional information is also required on background levels of nonpoint sources of pollution in forested areas especially with regard to seasonal variation.

The water quality modeling approach developed should be tested in other geographical areas before it can be accepted as a universal and a reliable model. As a result of these tests, areas requiring further development or refinement of the methodology or programming will be identified.

The SWMM program structure should be modified to allow analysis of at least 10 pollutants. Although this was accomplished by this task for the Runoff Block the other program blocks of the SWMM need to be modified so that all pollutants generated from a watershed can be transported in one run of the model.

The use of the modified SWMM should be promoted so that other users may utilize the vastly improved capabilities and flexibility of the model. The use of baseflow recessions in designing for low frequency or dry weather flows should be considered. Porous pavements should be evaluated, by means of the modified SWMM, as urban runoff control facilities. The reduction of data requirements for modeling natural streams should be emphasized and the use of the modified SWMM in modeling natural drainage systems should be encouraged.

Water quantity and quality sampling at The Woodlands should be continued at least until the watersheds have stabilized and erosion rates are reduced to pre-development or uniform levels. Data collection should also be continued at the porous pavement area and a sufficient number of overflow events should be sampled so that the porous pavement model can be tested in detail under real world conditions. Also water quality sampling of the outflow from the drain as well as the water in storage should be continued during dry periods so that operation of the porous pavement system can be properly evaluated and changes in water quality may be understood.

Sedimentation surveys of all stormwater detention reservoirs should be performed at regular intervals and preferably after every major storm event. The sediment accumulation rates in the reservoirs would be very helpful in not only modeling runoff and water quality but also in determining the life expectancy of the reservoir.

Records of all construction activity should be maintained so that the location and total area under construction during a storm event will be known. This record will prove very helpful in modeling water quality from each watershed.

The modeling of erosion in the SWMM needs to be refined. The coefficients of the Universal Soil Loss Equation were derived for agricultural areas and their applicability to urban and forested areas is limited. Possibly, a new approach may have to be considered. The significance of pollution from construction activity has generally been underestimated. Further study and methods to control erosion in urbanizing areas must be developed.

A significant portion of the effort expended during this project has been the setup and error correction of the original SWMM version. The SWMM has several users across the country who provide input as to improvements and corrections to the model. The University of Florida at Gainesville, Florida has been essentially a clearing house for the updates being made to the SWMM.

In order to minimize computer compatability problems, constant contact was maintained with the University of Florida which has been very responsive to suggested improvements and has already implemented many of the changes in order to make latter SWMM versions compatible on all computers. This has facilitated the setup of each new version of the model.

The problems were not limited to compatability conditions between different computers. The reason for each new version of the SWMM has been modifications and error corrections to the previous versions. These changes have affected the results obtained from the previous version of the SWMM as well as the data deck structure; and therefore require a complete reevaluation of previously completed work and the associated time and financial expenses. This situation became especially critical when the problem with infiltration computation discussed in Section 6 was discovered.

Several programming anomalies and errors were identified in the SWMM version used in this study, and subsequently corrected in later versions of the model. Specifically, the major problems include:

1. Erroneous values in the Transport Block of the SWMM concerning the depth of flow in certain types of conduits. For example, a circular conduit that would be flowing 41.5% full would be carrying only 4% of its total flow capacity.
2. In the Transport Block, input values describing a special type of channel would be read in over existing values stored in the program describing a trapezoidal section. This error prohibited using the trapezoidal section whenever a special channel was used.
3. Discovery of the existence of an undefined variable being referenced by the program. This caused the program to go into an infinite loop during some calculations until the time limit was exceeded. Considerable time was lost in tracing the variable down so that corrective action could be taken.
4. Erroneous graphs would appear on the rainfall hyetograph when more than one rain gage was specified.
5. As described in Section 6, amount of infiltration was dependent on the time of start of the storm after the start of modeling. This meant that infiltration would be greater if the storm began at the start of modeling time rather than a few hours later.
6. Length of integration timestep must be shorter than rainfall timestep or rainfall values are in error. This is due to the program's method of averaging rainfall intensities.
7. The SWMM Version II manual specified normalized area increments but the program needed normalized depth increments. This error was discovered only after abnormal results were observed in several runs.

The rest of the errors encountered were minor format corrections.

All the corrections mentioned, except for the infiltration and the timestep changes, were included in the February 1975 release of the SWMM. The University of Florida has subsequently included the infiltration changes in the May 1976 release of the SWMM.

Additional debugging of the SWMM is necessary. A significant debugging effort and review of the program code has resulted as an adjunct to this study; but a comprehensive correction of program errors, a time consuming and sometimes frustrating process, was beyond the scope of this project.

SECTION 4

GENERAL PROJECT INFORMATION

THE STORM WATER MANAGEMENT MODEL

The Storm Water Management Model (SWMM) was developed in 1971 by the University of Florida and the U.S. Environmental Protection Agency. The SWMM is composed of five integral computation blocks as shown in Figure 1.

The Executive Block controls all activity within the model because all input/output functions for the other blocks are programmed into the Executive Block. The Runoff Block computes the quantity and quality of runoff for a given storm and stores the results in the form of hydrographs and pollutographs at the inlets to the main sewer system. The Transport Block sets up initial flow and infiltration conditions and then performs flow quantity and quality routing to produce combined flow hydrographs and pollutographs for the total drainage basin and at selected intermediate points. The quantity and quality of flow is stored and treated by predefined criteria in the Storage Block. The dispersion effects of the discharge in the receiving body of water are computed in the Receiving Water Block. A more detailed description is available in the SWMM User Manual Version II (2).

In general only one or two computational blocks as well as the Executive Block are used in a run but all blocks may be run together. The use of independent computation blocks allows for the examination of intermediate results. Implementation of the SWMM requires a computer having core storage capacity of at least 350K bytes which translates to high costs per run, which in turn could limit the number of options to be analyzed.

The data requirements to model an urban watershed are listed in Table 1. Line printer tabulations and specified hydrographs and pollutographs are predicted as output from the program.

Since its original release, the SWMM has undergone several major revisions; the most recent version became available in November 1977. The February 1975 version of the SWMM was used most

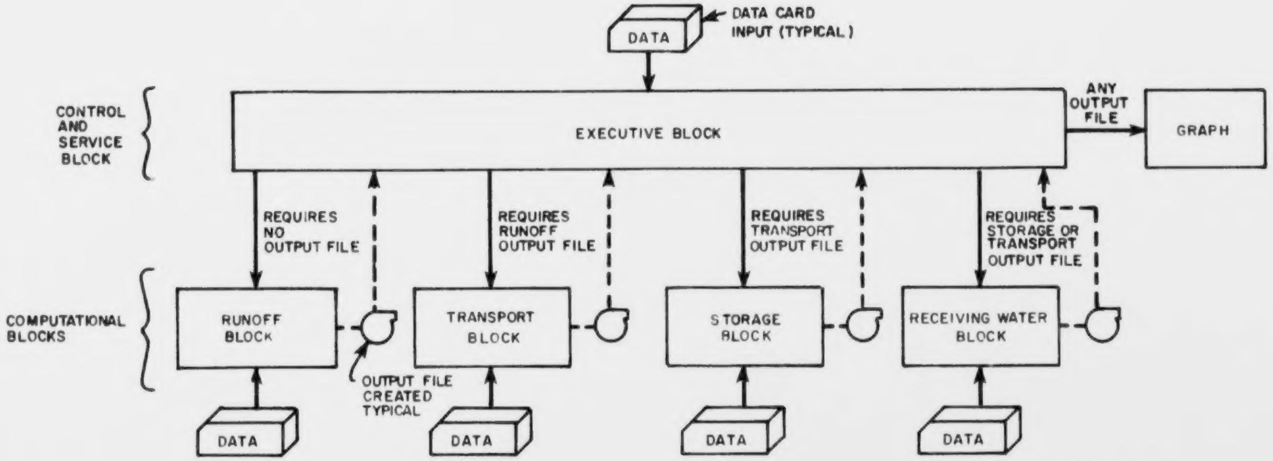

DATA
DATA CARD INPUT (TYPICAL)
CONTROL AND SERVICE BLOCK
EXECUTIVE BLOCK
ANY OUTPUT FILE
GRAPH
REQUIRES NO OUTPUT FILE
REQUIRES RUNOFF OUTPUT FILE
REQUIRES TRANSPORT OUTPUT FILE
REQUIRES STORAGE OR TRANSPORT OUTPUT FILE
COMPUTATIONAL BLOCKS
RUNOFF BLOCK
TRANSPORT BLOCK
STORAGE BLOCK
RECEIVING WATER BLOCK
OUTPUT FILE CREATED TYPICAL
DATA
DATA
DATA
DATA

Fig. 1 Master programming routine in the SWMM

TABLE 1

MODELING REQUIREMENTS BY SWMM

Item 1. Define the Study Area

Land use, topography, population distribution, census tract data, aerial photos, area boundaries.

Item 2. Define the System

Furnish plans of the collection system to define branching, sizes, and slopes. Types and general locations of inlet structures.

Item 3. Define System Specialties

Flow diversions, regulators, storage basins.

Item 4. Define System Maintenance

Street sweeping (description and frequency). Catch-basin cleaning. Trouble spots (flooding).

Item 5. Define the Receiving Waters

General description (estuary, river, or lake). Measured data (flow, tides, topography, water quality).

Item 6. Define the Base Flow (DWF)

Measured directly or through sewerage facility operating data. Hourly variation and weekday vs. weekend. DWF characteristics (composited BOD and SS results). Industrial flows (locations, average quantities, quality).

Item 7. Define the Storm Flow

Daily rainfall totals over an extended period (6 months or longer) encompassing the study events. Continuous rainfall hyetographs, continuous runoff hydrographs, and combined flow quality measurements (BOD and SS) for the study events. Discrete or composited samples as available (describe fully when and how taken).

from: SWMM Volume 1 - Final Report, July 1971 extensively during this study, and all modifications to the SWMM by this project have been incorporated into this version.

All modifications are included as user options so that the basic integrity of the SWMM is retained. Some of the new computational methods are dependent on site specific data.

STUDY OBJECTIVES

In general terms, the primary objectives of Espey, Huston and Associates, Inc. as part of the team involved in the overall study of the natural drainage system at The Woodlands were to modify and expand the capabilities of the SWMM and apply it to The Woodlands site. With the data on storm runoff quantity and quality which were collected by Rice University and the U. S. Geological Survey at The Woodlands, the model was to be used to evaluate the effectiveness of natural drainage systems in minimizing changes in storm runoff quantity and quality and to assist the engineers and planners in designing the drainage system for future phases of development at The Woodlands.

In specific terms, the study objectives of Espey, Huston and Associates, Inc. were:

1. Modify the SWMM as follows:
    a. to include a separate storm water system
    b. to reflect the interaction between groundwater conditions and surface drainage
    c. to reflect natural drainage concepts
    d. to include the cost of natural drainage systems
    e. to include the additional water quality parameters COD, Kjeldahl nitrogen, nitrates, and phosphates
    f. to include the effects of porous pavement.
2. Apply the SWMM to a developed Houston watershed as a prelude to modeling The Woodlands.
3. Apply the SWMM to Phase I of The Woodlands which is now under construction.
4. Use the SWMM to assist in the planning and development of the next development area at The Woodlands.

STUDY APPROACH

The scope of this study covered a period of three years. Initially the existing SWMM was evaluated by application to the Panther Branch and Hunting Bayou Watersheds. Hydrographs were developed for the following storms on Panther Branch, Hunting Bayou, and Swale 8, which is tributary to Panther Branch, in

Phase I of The Woodlands:

Panther Branch	Hunting Bayou	Swale 8
10/28/74	9/08/68	4/08/75
11/10/74	9/17/68	
11/24/74	11/05/68	
12/05/74	10/22/70	
12/10/74	11/09/70	
	3/26/74	
	5/08/75	
	6/30/75	

Attempts to model several other storms were abandoned due to data errors or extreme flow conditions.

The study objectives were accomplished in 12 tasks generally divided into 3 categories as follows:

Evaluation of the SWMM

Task 1. **Model observed storms on Hunting Bayou in Houston**

Task 2. Model observed storms on Panther Branch in The Woodlands

Task 3. Model Swale 8 in the developing area of The Woodlands

Modifications to the SWMM

Task 4. Modify SWMM to be used to model areas served by separate sewers including natural drainage

Task 5. Model the interaction between groundwater and surface drainage

Task 6. Improve the modeling of infiltration to allow periods of no rainfall

Task 7. Develop a subroutine to prepare area-discharge data for natural sections

Task 8. Develop a subroutine to compare costs of natural drainage relative to costs of conventional drainage

Task 9. Develop a methodology to determine predictive relationships for COD, Kjeldahl nitrogen, nitrates and phosphates and include modeling of these parameters in the SWMM

Task 10. Model the effects of porous pavements on rainfall and runoff relationships

Testing of the Modified SWMM

Task 11. Apply the modified SWMM to Hunting Bayou, Panther Branch and Swale 8 to observed events

Task 12. Apply the modified SWMM to Swale 8 to model future development trends

In accomplishing these tasks, several new subroutines were developed and incorporated into the SWMM. Each of the new subroutines is discussed in Sections 6 and 7 and listings of the program code are included in Appendix A. A revised set of input data coding instructions to allow the use of the new subroutines is also attached as Appendix B. The computer runs performed during this study were too numerous and were therefore not included in this report. All modeling input data are displayed in appropriate tables and figures in Section 8. All water quality data used in this study are summarized in tables in Appendix C. Appendix D contains hydrograph recession data; sample output from a SWMM run, utilizing all new computational schemes, is shown in Appendix E.

SECTION 5

STUDY AREA DESCRIPTION

HUNTING BAYOU WATERSHED

The Hunting Bayou Watershed is located on the eastern side of Houston near the intersection of Highways 59 and 610, as shown in Figure 2. The area is characterized by extremely mild land slopes (typically 0.1%) and impermeable soils with high clay content. The area receives approximately 114 centimeters (45 inches) of rain annually. Ninety-four percent of the total 800 hectare (1976 acre) area of the watershed is developed. Land usage is varied, with significant amounts of land being devoted to single and multi-family residential (48%), commercial (32%), and industrial (14%) purposes. The area as a whole has 21% impervious cover. It contains very few storm sewers with the vast majority of the area being drained by roadside grass-lined swales. The two main channels of Hunting Bayou are essentially trapezoidal with low flow channels. The banks and channel beds are earthen and are lined with vegetation which varies in density from moderate to very heavy depending upon the season of the year and maintenance schedules.

Inasmuch as these characteristics are also representative of the drainage system of The Woodlands, it was felt that this area would provide adequate calibration of the hydraulic portion of the SWMM. Most of the area is, in general, poorly maintained and the stream channels are sometimes used as dumping areas for waste materials such as oil and grease, old tires and other refuse. Because of this the quality of storm runoff in the area was significantly poorer than The Woodlands storm runoff.

The area modeled on Hunting Bayou, as shown in Figure 3, is the entire watershed upstream of the U. S. Geological Survey (USGS) gaging station No. 08075760 at Falls Street (Station H-20). This station, which has been in operation since 1964, continuously records flood hydrographs and rainfall. In the past, there has also been a gaging station upstream (USGS No. 08075750) at the Cavalcade Street Bridge on the south tributary of Hunting Bayou (Station H-10). This station was discontinued in 1973. Both of these stations are shown in Figure 3.

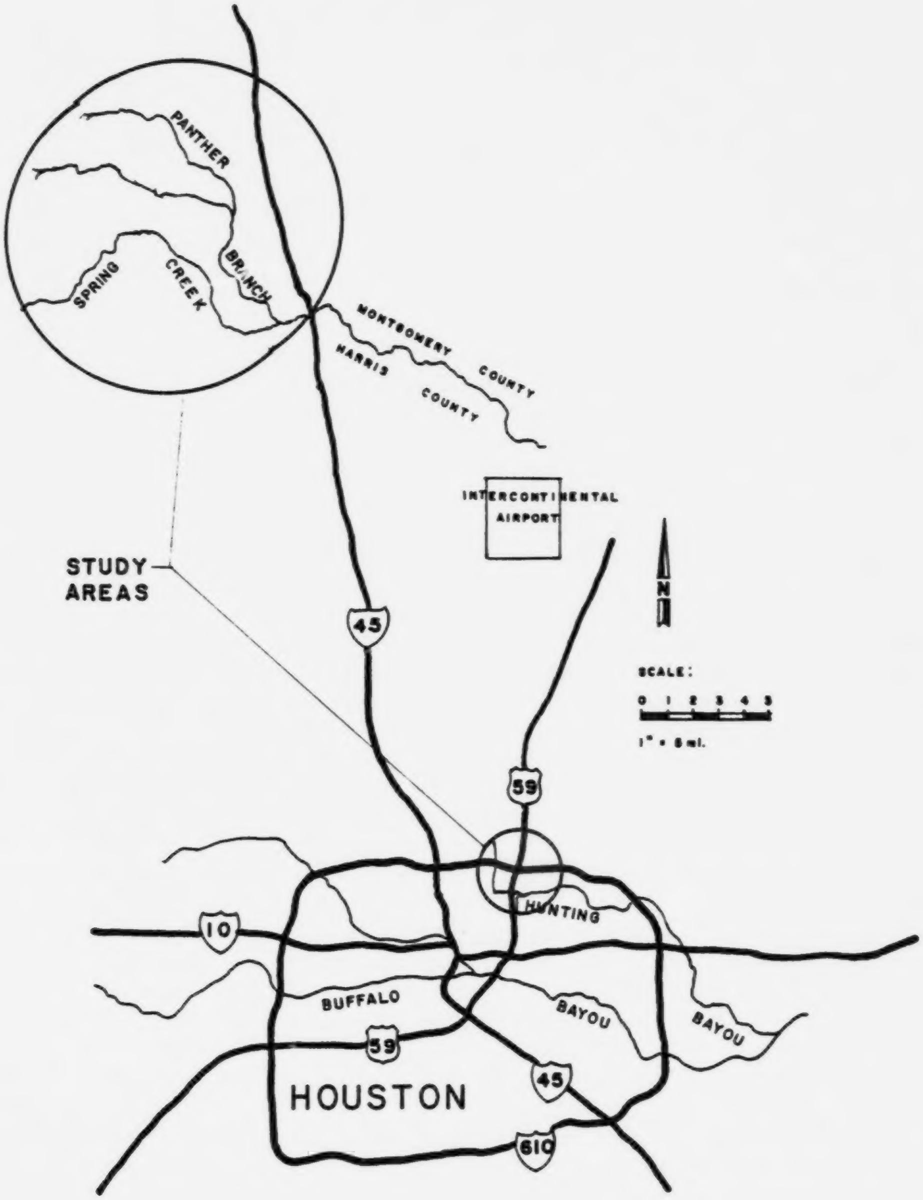

PANTHER
BRANCH
SPRING
CREEK
MONTGOMERY COUNTY
HARRIS COUNTY
INTERCONTINENTAL
AIRPORT
STUDY
AREAS
N
SCALE:
0 1 2 3 4 5
1" = 5 mi.
45
59
10
HUNTING
BUFFALO
BAYOU
BAYOU
59
45
HOUSTON
610

Fig. 2 Study area vicinity map

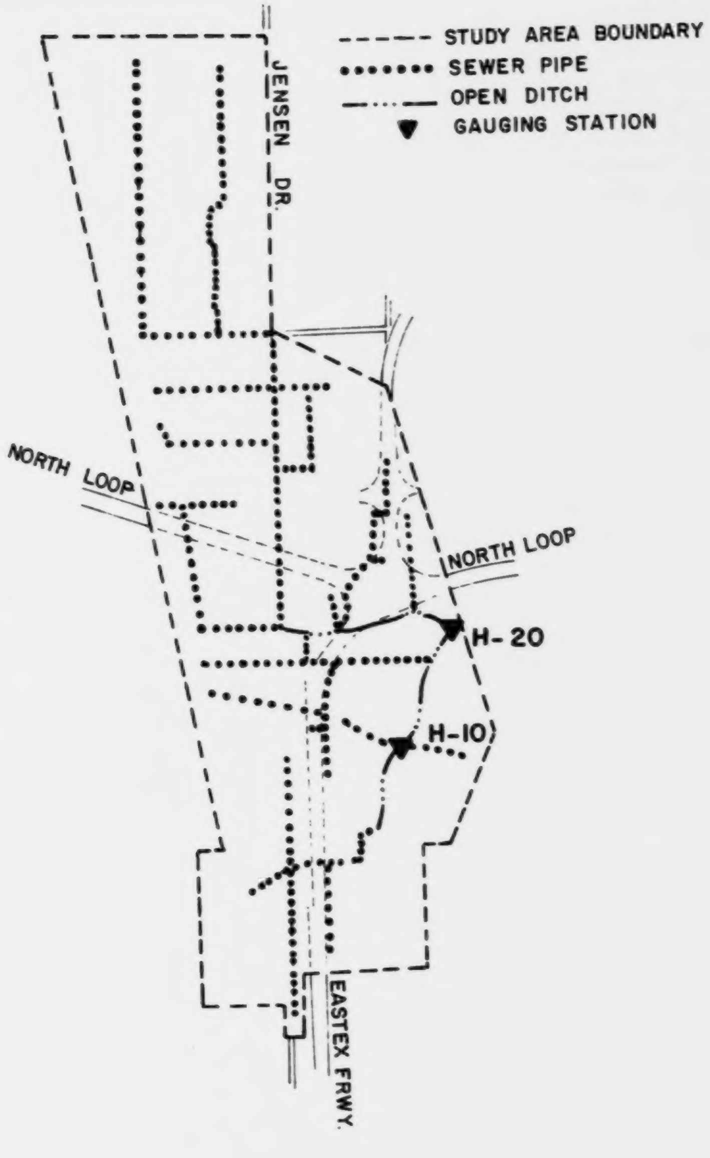

STUDY AREA BOUNDARY
SEWER PIPE
OPEN DITCH
GAUGING STATION
JENSEN DR.
NORTH LOOP
NORTH LOOP
H-20
H-10
EASTEX FRWY.

Fig. 3 Hunting Bayou watershed

THE WOODLANDS DEVELOPMENT AND PANTHER BRANCH WATERSHED

The Woodlands is a new community being developed in Montgomery County, Texas. The community is situated in a heavily forested tract about 45 kilometers (28 miles) north of Houston. The Woodlands encompasses 7,200 hectares (17,800 acres) (Figure 4) and is planned to be developed over a twenty-year period, which began September 1972. A total of 33,000 dwelling units are programmed with a projected population of 112,000 in 1992. A concern for nature and convenience for man were two of the major criteria used in the development of the General Plan for The Woodlands. The basis for all aspects of development in The Woodlands was a unique ecological inventory conducted from 1971 to 1973.

The Woodlands site is located in the Spring Creek drainage basin. The major stream draining The Woodlands is Panther Branch which is a tributary of Spring Creek. Panther Branch is an intermittent stream with major no-flow periods occurring during the summer months. A flow gaging station, P-30 (USGS No. 08068450) was established near the lower end of The Woodlands site in May, 1972. The drainage area upstream of the gage is 54.39 square kilometers (33.8 square miles).

Panther Branch and its tributary Bear Branch form the principal drainage system upstream of the developing areas in The Woodlands (Figure 4). A stream stage recorder, Station P-10, is located below the confluence of Bear and Panther Branches and has a drainage area of 24.30 square kilometers (15.10 square miles). The drainage area of P-10 is undeveloped forest land while the P-30 drainage area includes Phase I development of The Woodlands.

The basic drainage system planned for The Woodlands has been designed on the basis of what has been termed the natural drainage concept. This concept consists of the following principles: 1) The existing drainage system in its unimproved state is utilized to the fullest extent possible; 2) Where drainage channels need to be constructed, wide shallow swales lined with existing native vegetation are used instead of cutting narrow, deep ditches; 3) Drainage pipes and other flood control structures are used only where the natural system is inadequate to handle increased urban runoff, such as in high-density urban activity centers, and 4) Flow retarding devices such as retention ponds, recharge berms and porous pavements are used where practical to minimize increases in runoff volume and peak flow rates due to development. The natural drainage concept as outlined by these four principles seeks to minimize changes in the runoff regime due to urbanization by providing increased infiltration and storage capacity and higher resistance to flow within the channels.

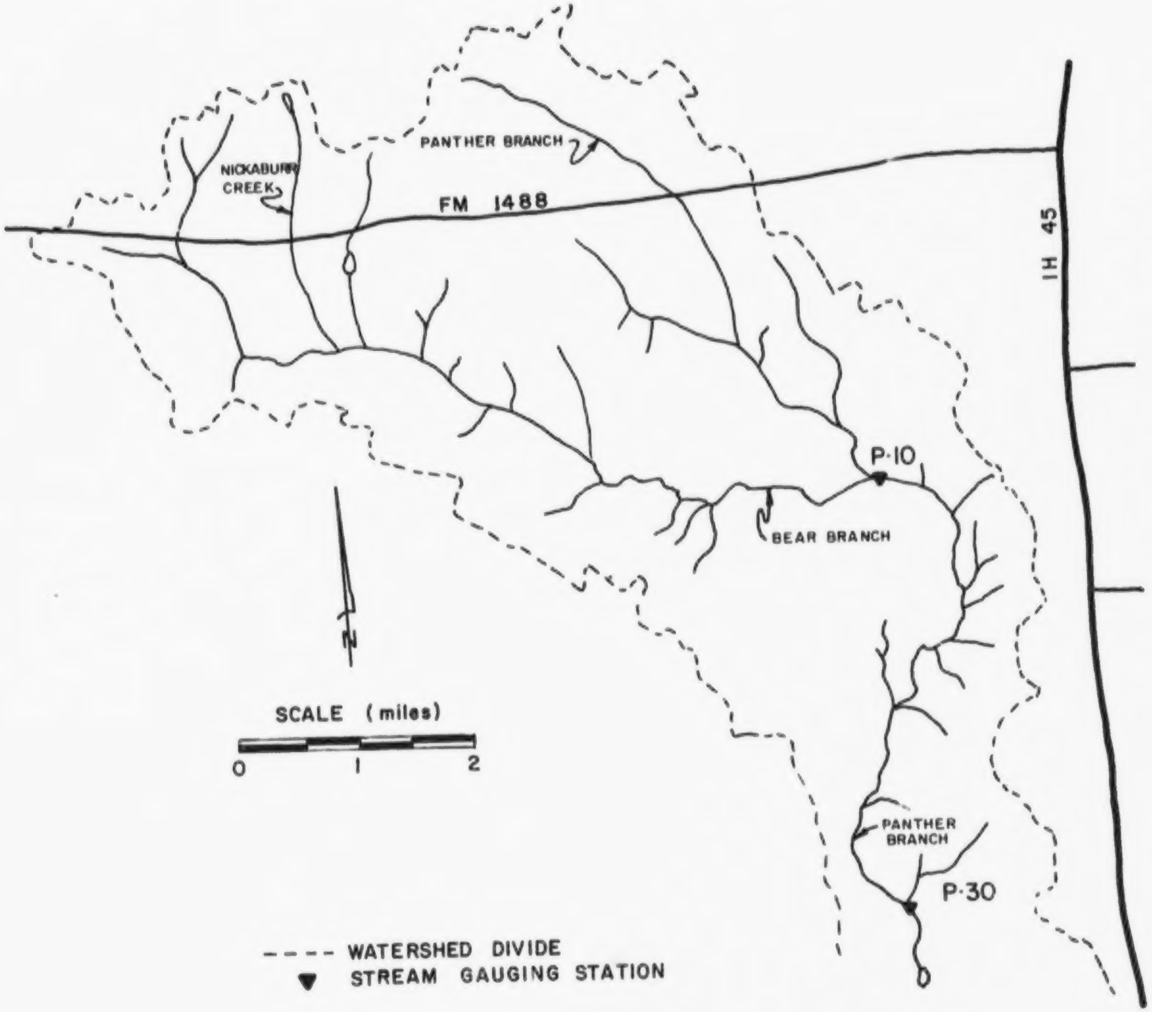

PANTHER BRANCH
NICKABURR CREEK
FM 1488
IH 45
P·10
BEAR BRANCH
N
SCALE (miles)
0
1
2
PANTHER BRANCH
P·30
- - - - WATERSHED DIVIDE
▼ STREAM GAUGING STATION

Fig. 4 Panther Branch watershed

SWALE 8 WATERSHED

The first area to be developed at The Woodlands is referred to as Phase I and is located immediately upstream from Station P-30. The largest individual drainage area in Phase I is Swale 8. With a drainage area of 195.46 hectares (483 acres), Swale 8 drains into Panther Branch through Lakes A and B.

As shown in Figure 5, the downstream third of the watershed, west of Grogans Mill Road, is substantially urbanized primarily in commercial and multi-family residential development.

The drainage system for Swale 8 was designed on the basis of the natural drainage concepts described earlier. In conjunction with the natural drainage system, three major reservoirs are located in the Swale 8 drainage area. Two reservoirs (Lakes A and B), approximately 4.8 and 3.0 hectares in size (12 and 8 acres), respectively, are located at the Conference-Leisure Center. The 3 hectare reservoir will empty directly into the 4.8 hectare reservoir. The normal operating level of the larger reservoir will be held at 1 meter (3 feet) below the storm outflow depth thereby providing 1 meter of flood-control storage. The effects of this storage are very significant in reducing the impact of urbanization on the quantity and quality of runoff from Swale 8 downstream from the lake. Lake C, a 2.8 hectare (7 acre) reservoir on the east side of Grogans Mill Road has also been completed recently. One meter (3 feet) of storage is provided in it for runoff control.

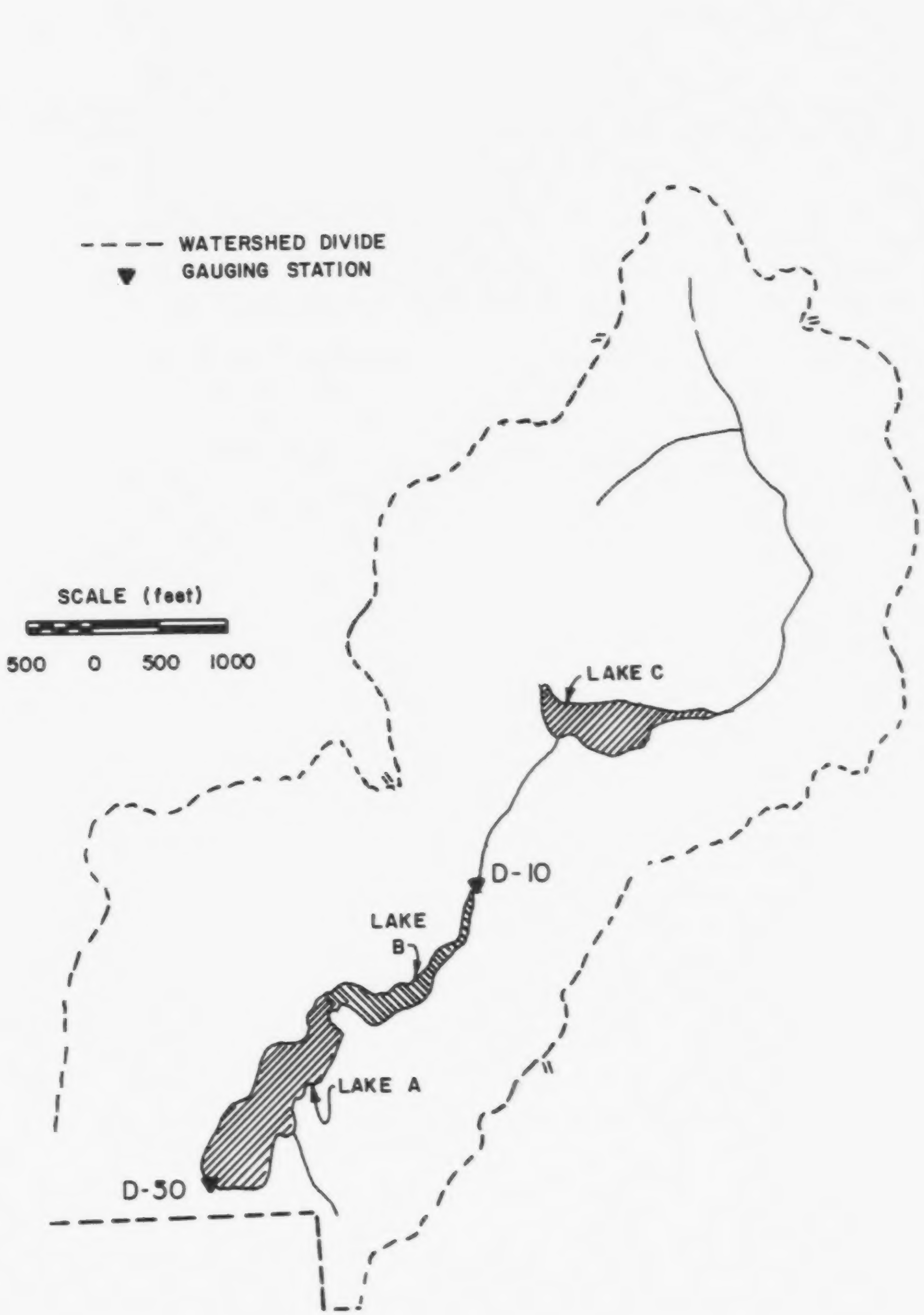
Two stream gaging stations are operated by the U. S. Geological Survey in the Swale 8 watershed. Station D-50 measures the outflow from Lake A and Station D-10, located on the east side of Grogans Mill Road, measures the inflow into Lake B from Swale 8.

DATA COLLECTION

As described previously, all stream flow stations are maintained by the USGS. The continuous records at these stations proved very useful during this study; the period of record for each station is as follows:

P-10	10/73 to present
P-30	4/72 to present
H-10	4/64 to 9/73
H-20	4/64 to present
D-10	10/74 to present
D-50	11/74 to present

Monthly grab samples for water quality were also collected by the USGS.



WATERSHED DIVIDE
GAUGING STATION
SCALE (feet)
500 0 500 1000
LAKE C
D-10
LAKE B
LAKE A
D-50

Fig. 5 Swale 8 watershed

During the initial phases of this project, several problems with a discharge rating curve for Station P-10 were experienced; but a satisfactory rating curve was finally developed. The long processing time to convert gage heights to discharge rates was substantially reduced by the development of a computer rating curve program. The computer program, with the aid of an input rating curve would convert gage heights to discharge rates from which the storm hydrograph was developed.

The temporal and spatial distributions of several key water quality parameters were defined for individual storm events through comprehensive sampling by Rice University. Parameters analyzed included but were not limited to suspended solids, COD, Kjeldahl nitrogen, nitrates, and phosphates. Because of the processing time, these data were not available immediately, resulting in a lag of several months between data sampling and water quality modeling. However, all the data had been processed and were available prior to the end of the project. A summary of all storms sampled by Rice University is listed in Table 2.

TABLE 2. STORM EVENT HYDROLOGY SUMMARY

#	Date	Site	Rainfall, inches	Total Stream-flow, acre-ft[a]	Base Flow, CFS	Peak Flow, CFS	Runoff, acre ft
1	1/18/74	Woodlands P-30	2.02	2334.0	40.0	1260.0	2131.0
2	3/20/74	Hunting Bayou	0.15	2.507	5.0	9.6	0.857
3	3/26/74	Hunting Bayou	0.75	42.06	9.5	40.0	24.8
4	4/11/74	Hunting Bayou	0.35	12.41	4.0	11.0	5.47
5	4/22/74	Woodlands P-30	0.45	5.8	0.2	9.7	5.39
6	10/28/74	Woodlands P-30	3.46	937.0	1.6	382.0	928.4
7	12/05/74	Woodlands P-30	1.59	1267.0	4.5	332.0	1238.0
		Woodlands P-10	1.52	833.0	2.1	280.0	822.0
8	3/04/75	Woodlands P-30	0.33	3.318	1.75	4.9	1.8
		Lake A	0.26	No Discharge	No Discharge		-
		Lake B	0.26	0.135	0.0	0.38	0.135
9	3/13/75	Woodlands P-30	0.75	119.0	1.3	49.0	111.0
		P-10	0.69	79.09	0.64	56.0	77.0
		Lake A	0.81	2.38	0.0	2.0	2.38
		Lake B	0.81	1.77	0.0	12.7	1.77
10	4/08/75	Woodlands P-30	2.76	2829.0	0.48	1100.0	2826.0
		P-10	2.43	1614.0	1.0	1170.0	1610.0
		Lake A	3.97	93.39	0.0	114.0	93.4
		Lake B	3.97	93.2	0.0	123.0	93.2
11	5/08/75	Hunting Bayou	0.81	28.9	4.6	72.5	23.8
		Westbury	0.75	7.16	0.0	36.0	7.16
12	06/30/75	Hunting Bayou	1.85	115.7	14	136	94.0
		Westbury	1.28	11.23	0	47	11.23
13	09/05/75	Woodlands P-30	.35	9.825	0.14	10.3	9.48
		Woodlands P-10	.25	0.918	0.02	0.38	0.822
		Lake A	1.11	No Discharge	0	---	0
		Lake B	1.11	1.51	0	8.8	1.51
14	10/25/75	Woodlands P-30	2.89	117.35	0	64	117.35
		Woodlands P-10	2.82	57.09	0	22	57.09
		Lake A	3.37	18.86	0	15	18.86
		Lake B	3.37	No Data	-	-	-
15	03/07/76	Woodlands P-30	.713	14.88	1.45	18.5	12.6
		Woodlands P-10	.68	11.24	.3	7.9	10.37
		Lake A	.69	No Discharge	-	-	6.5
		Lake B	.69	0.884	0	3.8	.884
16	03/08/76	Woodlands P-30	.53	99.3	8.4	32.5	59.0
		Woodlands P-10	.48	58.3	5.3	23	29.1
		Lake A	.71	6.78	0	3.4	6.78
		Lake B	.71	2.71	0	6.6	2.71
17	04/04/76	Woodlands P-30	.366	45.95	.18	64	45.56
		Woodlands P-10	.29	.764	.11	.37	0.537
		Lake A	1.17	14.54	0	10	14.53
		Lake B	1.17	5.10	0	19	5.103

[a] Total Streamflow is calculated to include components of overland runoff and base flow.

[b] Percentage of rainfall as runoff.

SECTION 6

MODIFICATIONS TO THE SWMM

MODELING A SEPARATE STORM WATER SYSTEM

The SWMM was originally developed for drainage systems which included combined sewer systems. In the case of separate sewer systems, it was necessary to model urban stormwater systems which do not include the dry weather flow (DWF) component of combined sewer systems. Consequently, subroutine INFIL, to compute infiltration into the sewer, and subroutine FILTH, which estimates DWF based on population, were not called in the Transport Block and Card Groups 30 through 44 were omitted in all runs, thus providing a considerable reduction in input data requirements.

In modeling natural drainage systems, the lack of catch basins allowed a further omission of input data. Card Group 17, catch basin data, in the Runoff Block data deck was a blank card.

The modifications to the SWMM to model a separate storm water system are relatively simple and no difficulties were encountered with this phase of the project.

INTERACTION BETWEEN GROUNDWATER CONDITIONS AND SURFACE DRAINAGE

In the absence of storm sewers and their associated infiltration rates in a natural drainage area, hydrograph recessions were used as indicators of the interaction between groundwater conditions and surface drainage. Several investigators including Holtan and Lopez (3) consider hydrograph recession rates to be a function of total volume of water in storage in surface depressions, vegetative and soil layers and in groundwater. The rate at which these storage volumes are drained is determined to be the recession rate of the hydrograph beyond the point of inflection when all surface runoff is assumed to have ceased (4).

The physical process of draining water from storage can be approximated by a linear reservoir whose outflow rate is a function of storage. This relationship is defined by Riggs (5) and others as follows:

$$q_t = q_o K^t$$

where

q_o is the discharge at some initial time

q_t is the discharge at any instant

K is the recession constant

t is the number of time units elapsed since q_o

In the above equation, the numerical value of K depends on the time unit selected. Consequently, the equation was redefined as follows:

$$q_t = q_o\, e^{Kt}$$

In this case all units are as defined previously except K, which is now defined as $\frac{\Delta \ln q}{\Delta t}$. This equation can be linearly represented on a ln q versus t graph.

From studying a large number of observed hydrographs, Barnes (6) determined that the different outflow rates from water stored in depressions, surface soil, and groundwater could be approximated by 3 separate straight line functions on a semilogarithmic plot. Later studies (4,7) have substantiated this assumption and Figure 6 depicts this characteristic at Station P-10.

This approach was utilized in the inclusion of baseflow modeling by the SWMM in subroutine BASFLO. A maximum of 5 recession rates are allowed. Each recession rate was made a function of flow. User input data were developed by first transforming all discharge values for all hydrographs at P-10 and P-30 to the natural logarithmic values and then graphically determining the slope, K, on plots of ln q versus t for each hydrograph recession segment which indicated a linear relationship. Therefore, each recession is composed of a number of straight lines which begin at a specific ln q and end at another specific ln q as shown in Figure 6. The slope values were then graphed against the beginning ln q values and straight line equations of the form $K = Ko + m \ln q$ for ln q versus slope were derived by means of least squares analysis as shown in Figures 7 and 8. Because extreme data points are critical in least squares methods, all extreme data points were subjectively deleted prior to the derivation of the equations.

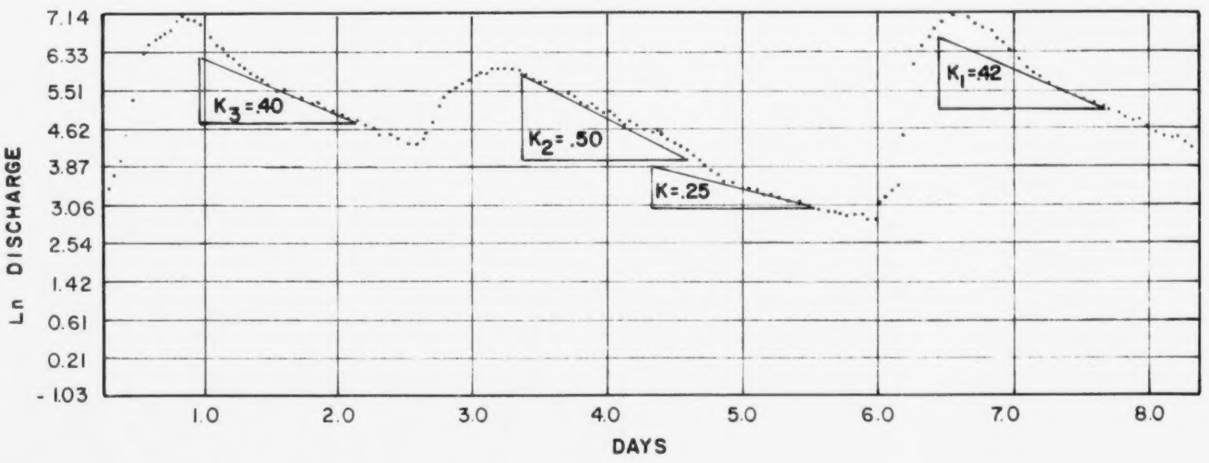


Fig. 6 Baseflow recession curves at Station P-10

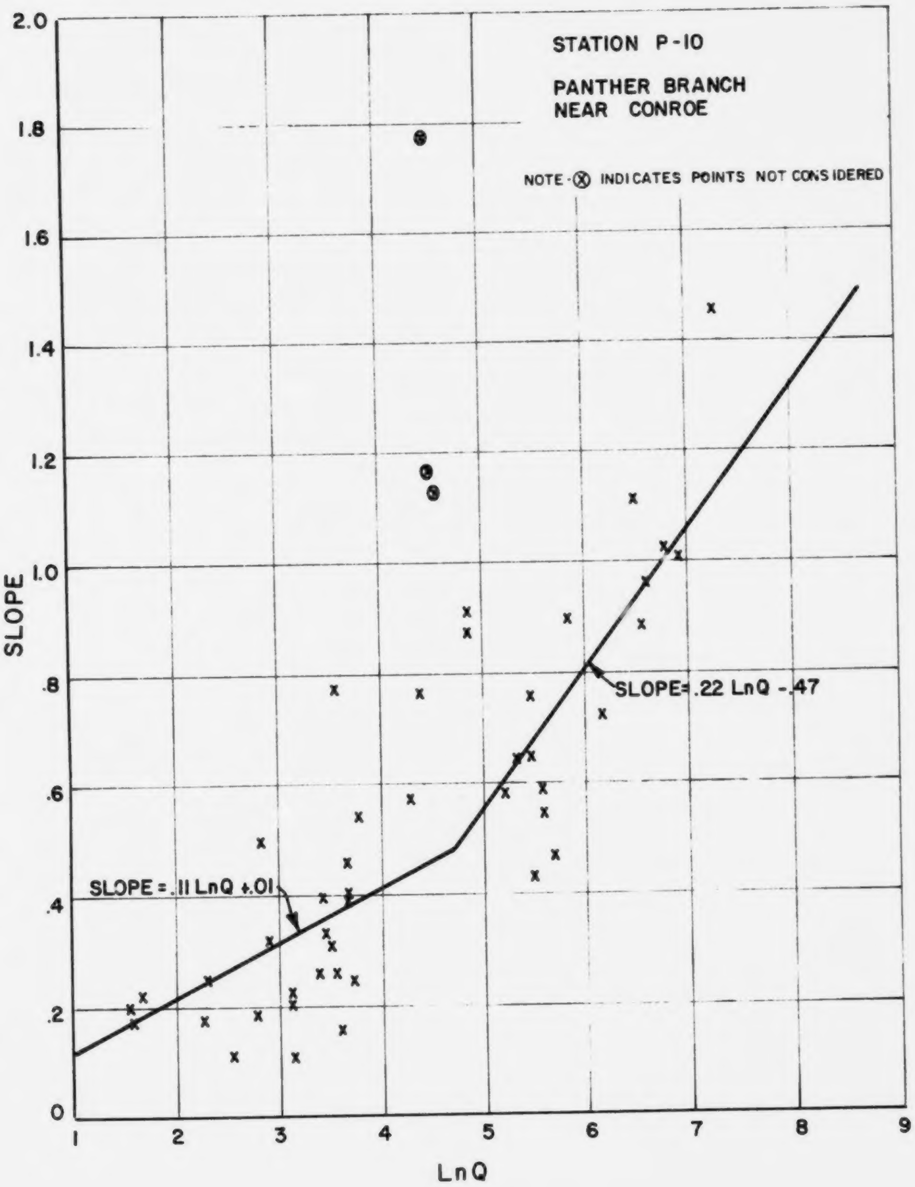


Fig. 7 Slopes of hydrograph recessions at Station P-10

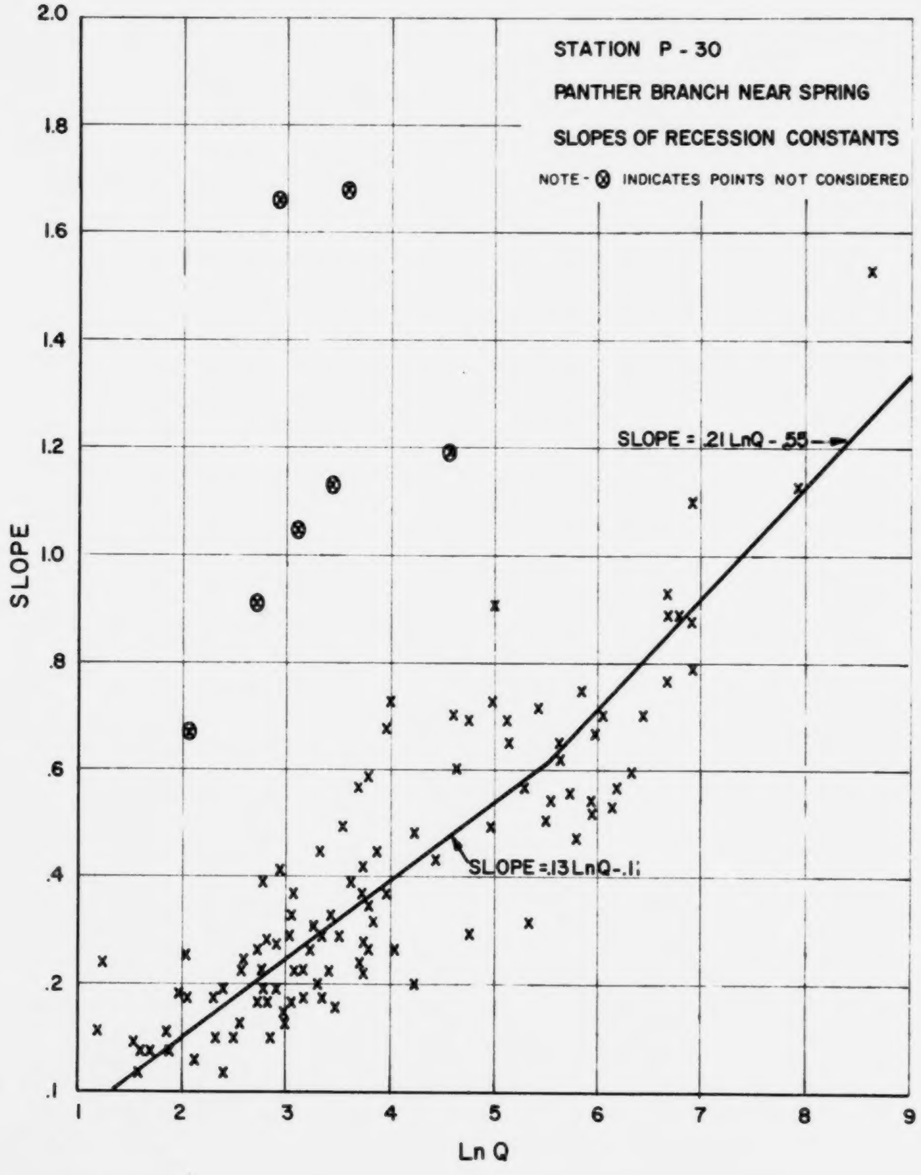


Fig. 8 Slopes of hydrograph recessions at Station P-30

Subroutine BASFLO requires input values of starting ln q and the slope, m, and intercept, Ko, coefficients to compute the recession rate, K. Five sets of data may be input.

A flow chart of all computations performed in Subroutine BASFLO is shown in Figure 9. In general, the hydrograph at the downstream end of the system, which is output from the Transport Block, is input to subroutine BASFLO which then performs the following sequential computations.

1) Consecutive flow rates are compared to determine the hydrograph peak.

2) For each flow rate, q_i, after the hydrograph peak, the slope with respect to the previous flow rate, q_{i-1}, is compared to the slope to the next flow rate, q_{i+1}. As long as the absolute values of the slopes are increasing with each time step, the computations proceed with no interruption. A decrease in the absolute values of the slopes indicates that the point of inflection has been located.

3) Using this flow rate as the beginning flow rate, subroutine BASFLO searches the input data table to determine during which recession interval the present recession begins and selects the corresponding recession rate coefficients.

4) The value of the recession rate, K, is computed and the recession equation applied to determine the new flow rate, q_t.

5) If no further surface inflow to the stream occurs as a result of a second rainfall, for example, the flow rates are receded into the domain of a new starting flow rate and a new set of recession rate coefficients.

6) This procedure is continued until the hydrograph starts rising again or until the computational time steps are exceeded.

The recession flow rates computed in subroutine BASFLO replace all flow rates after the point of inflection on the hydrograph and the resulting total storm hydrograph is output and graphed if so desired.

For streams which receive low flow contributions from groundwater aquifers, subroutine BASFLO includes an option to include these flow rates as a constant rate, a linearly varying rate, or an exponentially varying rate. The option to use

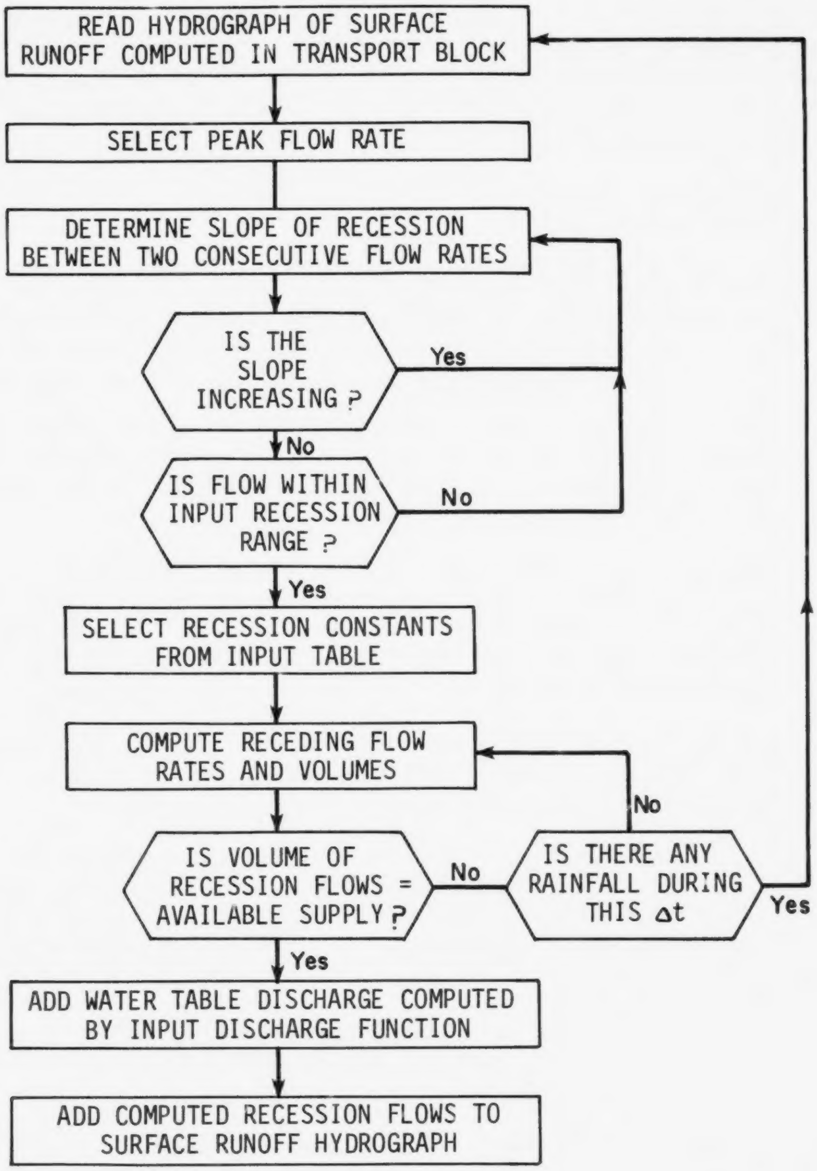

READ HYDROGRAPH OF SURFACE RUNOFF COMPUTED IN TRANSPORT BLOCK
SELECT PEAK FLOW RATE
DETERMINE SLOPE OF RECESSION BETWEEN TWO CONSECUTIVE FLOW RATES
IS THE SLOPE INCREASING ?
Yes
No
IS FLOW WITHIN INPUT RECESSION RANGE ?
No
Yes
SELECT RECESSION CONSTANTS FROM INPUT TABLE
COMPUTE RECEDING FLOW RATES AND VOLUMES
No
IS VOLUME OF RECESSION FLOWS = AVAILABLE SUPPLY ?
No
IS THERE ANY RAINFALL DURING THIS Δt
Yes
Yes
ADD WATER TABLE DISCHARGE COMPUTED BY INPUT DISCHARGE FUNCTION
ADD COMPUTED RECESSION FLOWS TO SURFACE RUNOFF HYDROGRAPH

Fig. 9 Subroutine BASEFLO system logic

varying rates is in recognition of the fluctuations sometimes observed in water tables, and especially in perched water tables as are found at The Woodlands.

The use of subroutine BASFLO is dependent on the availability of onsite or site compatible data for recession rates. If these data are not readily available the modeling accuracy is diminished.

Changes in the recession rates resulting from urbanization range from the elimination of depression storage by leveling and grading, to total prevention of groundwater recharge by use of an impervious pavement. These effects have not been quantitatively evaluated due to the deceleration of urbanization at The Woodlands. At a later date these data should become available, but until then, the user must supply recession rate reduction factors based only on engineering judgment or data from other sources.

The simulation of observed recessions at P-10 and P-30 for two storms each are shown in Figures 10 and 11. These results indicate a close correspondence with the observed events and this should be so because the recession rate coefficients to determine individual recessions were derived from all observed events at the same locations. The closeness of fit indicates that hydrograph recessions at P-10 and P-30 can indeed be approximated by average recessions.

At the initiation of this project, 6 surface soil water measuring wells were installed at different locations in The Woodlands but unfortunately the cost of maintenance on the sampling program was too high to be cost effective and consequently no data on the accretion or depletion of surface soil water are available.

A number of factors affect the uniformity as well as the reliability of recession curves; some of the major factors are listed below:

1. In areas with a marked vegetative growing season and in agricultural areas, marked variations in evapotranspiration losses result in recession rate variability. Therefore, data from streams in these areas should be carefully selected.
2. Channel and bank storage effects will affect the recession rates and must be considered when inter-basin transfer of data is necessary.
3. If more than one aquifer supplies water to a stream, the contribution from each aquifer must be considered separately.

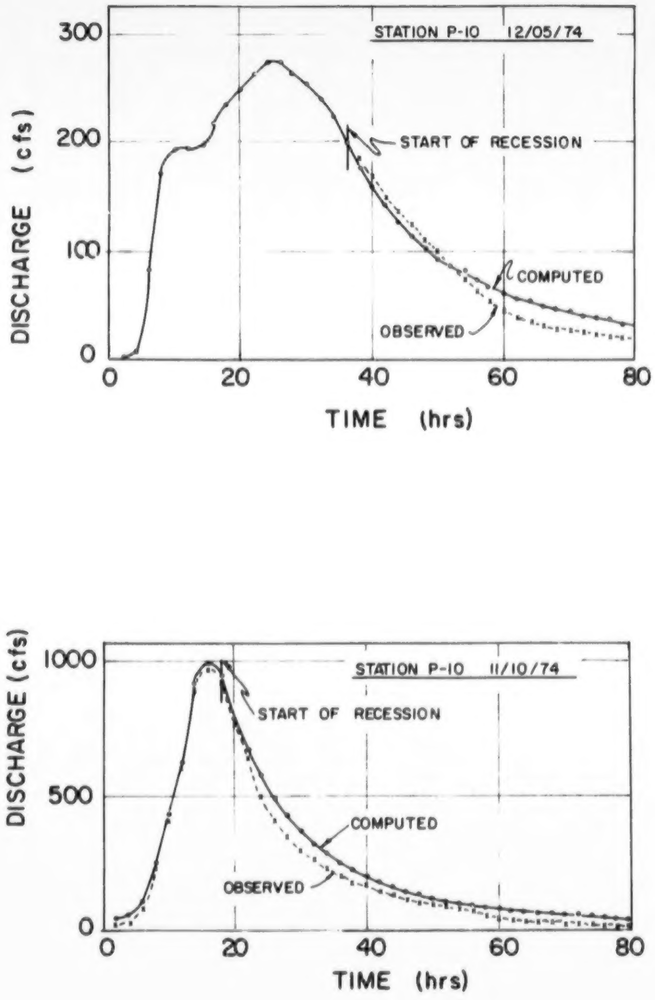


Fig. 10 Baseflow recessions at Station P-10

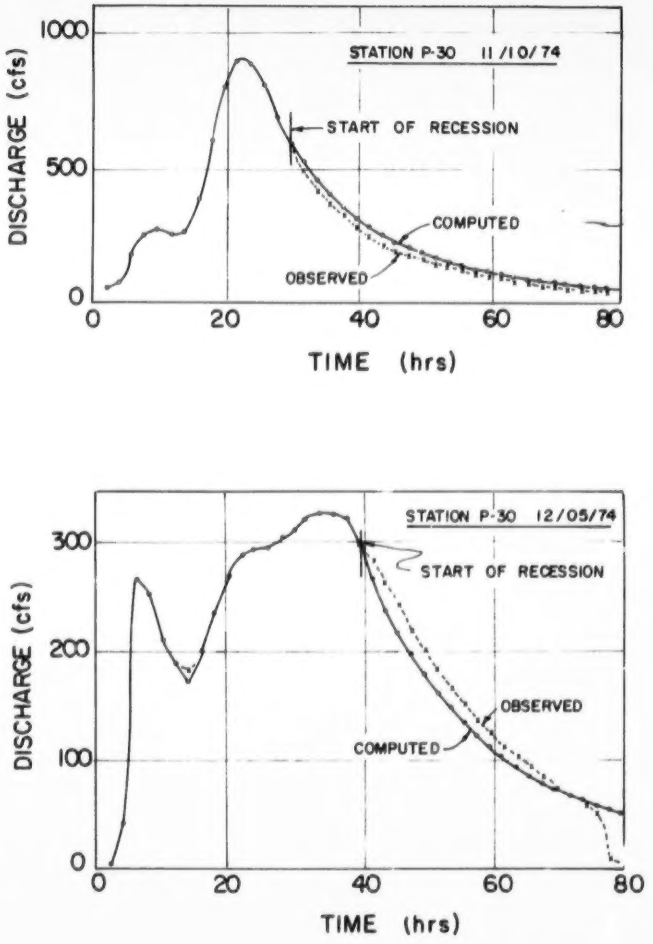


Fig. 11 Baseflow recessions at Station P-30

4. Snow melt rates, if applicable, should also be evaluated if baseflow recessions during snowmelt periods are being investigated.

INFILTRATION

Another aspect of the interaction between groundwater and surface drainage is the modeling of infiltration in the SWMM. Infiltration, which usually starts at a high rate and decreases during rainfall to a lower constant rate, occurs in a three step sequence: 1) entry of water through the soil surface, 2) transmission through the soil and 3) the filling of the storage capacity of the soil. The surface entry rate may be reduced by the inwashing of fines or other particles and by raindrop impact. Therefore, infiltration will be limited to the lowest transmittal rate encountered by the water. The available storage in a soil depends on the porosity and thickness of the soil horizon. The average infiltration capacity, f_c, during the time interval is taken to be the value at the center of the time interval $(t + 0.5\ \Delta t)$ and is calculated by Horton's (8) equation:

$$f_c = (f_i - f_o)e^{-kt^*} + f_o$$

where

f_c = infiltration capacity at time t^*

f_i = maximum infiltration rate

f_o = minimum infiltration rate

k = decay rate

t^* = time from the start of rainfall to the midpoint of the time interval = $t + 0.5\ \Delta t$

After calibration of the SWMM for a study watershed, the only input data that were varied were precipitation and the coefficients for Horton's infiltration equation, especially the starting infiltration rate. Horton's equation, a function of time only, has been found to fit a large number of experimental infiltration capacity curves obtained from different soils with different types of vegetal cover and in widely separated regions. The equation was derived from experimental infiltration tests in which the supply of water exceeds the infiltration capacity. Therefore, a limitation of the equation is that rainfall intensity must always exceed the infiltration capacity. In naturally occurring rainfall, the rainfall intensity can be less than the infiltration capacity and periods of zero rainfall can occur within a storm event.

In the infiltration model in the revised SWMM version after the input parameters (f_i, f_o, k) are initialized, the infiltration capacity decays as a function of the time elapsed from the start of computations. But the decay of the infiltration rate during a storm event of nonuniform intensity will be a function of present rainfall intensity, past rainfall, and time. In the SWMM, if the rainfall intensity is less than the infiltration capacity, the capacity will decrease at the constant decay rate (k) as if the rainfall intensity were equal to the infiltration capacity.

In applying the model it was found that increasing the maximum infiltration capacity in some cases had no effect on the volume of infiltration. A manual calculation through one time interval showed that the infiltration volume should have increased. Upon investigation, a programming anomaly was discovered. In the SWMM, the decay of infiltration capacity started at time zero, not at the start of rainfall. If rainfall started at four hours after the start of modeling, the infiltration capacity calculated by the SWMM would have decayed for four hours and will be at or near the minimum capacity at the actual start of rainfall. This may be the reason why previous sensitivity studies showed that varying the maximum infiltration rate produced no effect on the runoff volume (9).

The SWMM's infiltration model was modified by EH&A to use Horton's equation in an integrated form in association with a temporal parameter that follows the progress of infiltration on each subcatchment as described below. In the integrated form, Horton's equation is:

$$M = f_o t + (f_i - f_o)\ (1 - e^{-kt})$$

where M is the accumulated volume of water, in inches, infiltrated up to a time, t; and the other variables are the same as defined previously.

During each time interval, Δt, the volume of water the soil is capable of infiltrating, in inches, $(M_{t + \Delta t} - M_t)$ is calculated. This volume of water is compared to the total volume of water available for infiltration. When the available volume is greater than the infiltration volume,

$$D_1 > (M_{t + \Delta t} - M_t)$$

the excess is calculated and the results are comparable to the present infiltration model in SWMM.

If the infiltration volume is greater than the available volume,

$$M_{t + \Delta t} - M_t) > D_1$$

the time increment, $t_c < t$, is calculated such that the infiltration volume is equal to the available volume,

$$(M_{t + t_c} - M_t) = D_1$$

and runoff is set equal to zero. The time $(t + t_c)$ is then used as the starting point of infiltration in the next time interval.

The University of Florida has already incorporated this infiltration modeling scheme in the May 1976 version of the SWMM.

COSTS OF NATURAL DRAINAGE SYSTEMS

One of the major factors to be considered by engineers and planners in designing natural drainage systems is the relative cost of such systems with respect to conventional drainage systems which generally consist of sewer networks, improved and realigned channels. When the SWMM is being used for design purposes, the ability to determine these relative costs through Subroutine CSTANL could be very beneficial.

The description of the drainage system in the Transport Block of the SWMM is in two dimensions only; elevations and excavation depths are not used. Therefore, Subroutine CSTANL uses unit area costs to determine total project costs.

Unit area costs for natural and conventional sewered drainage systems and for right-of-way acquisition and clearing are user supplied, if available, or the default values based on 1970 costs at The Woodlands are used. Dated cost data are adjusted for present conditions by use of the Engineering News-Record Cost Index. These unit costs are applied to each subcatchment drainage area to determine the total costs for natural and conventional drainage systems for each subcatchment. The ratio of natural drainage costs to conventional drainage costs is also determined. This ratio may then be used to evaluate the desirability of either drainage system for each subcatchment.

A flow chart of computations in Subroutine CSTANL is shown if Figure 12. The computations proceed as follows:

1. Unit costs of conventional and natural drainage systems are input to the model. If these costs are not available, the default values of \$686/hectare (\$1700/acre) for conventional and \$121/hectare (\$300/acre) for natural systems are used. These values are based on 1970 data for The Woodlands.

2. If the cost data are not current, the Engineering News-Record Cost Index is utilized to update the cost data.

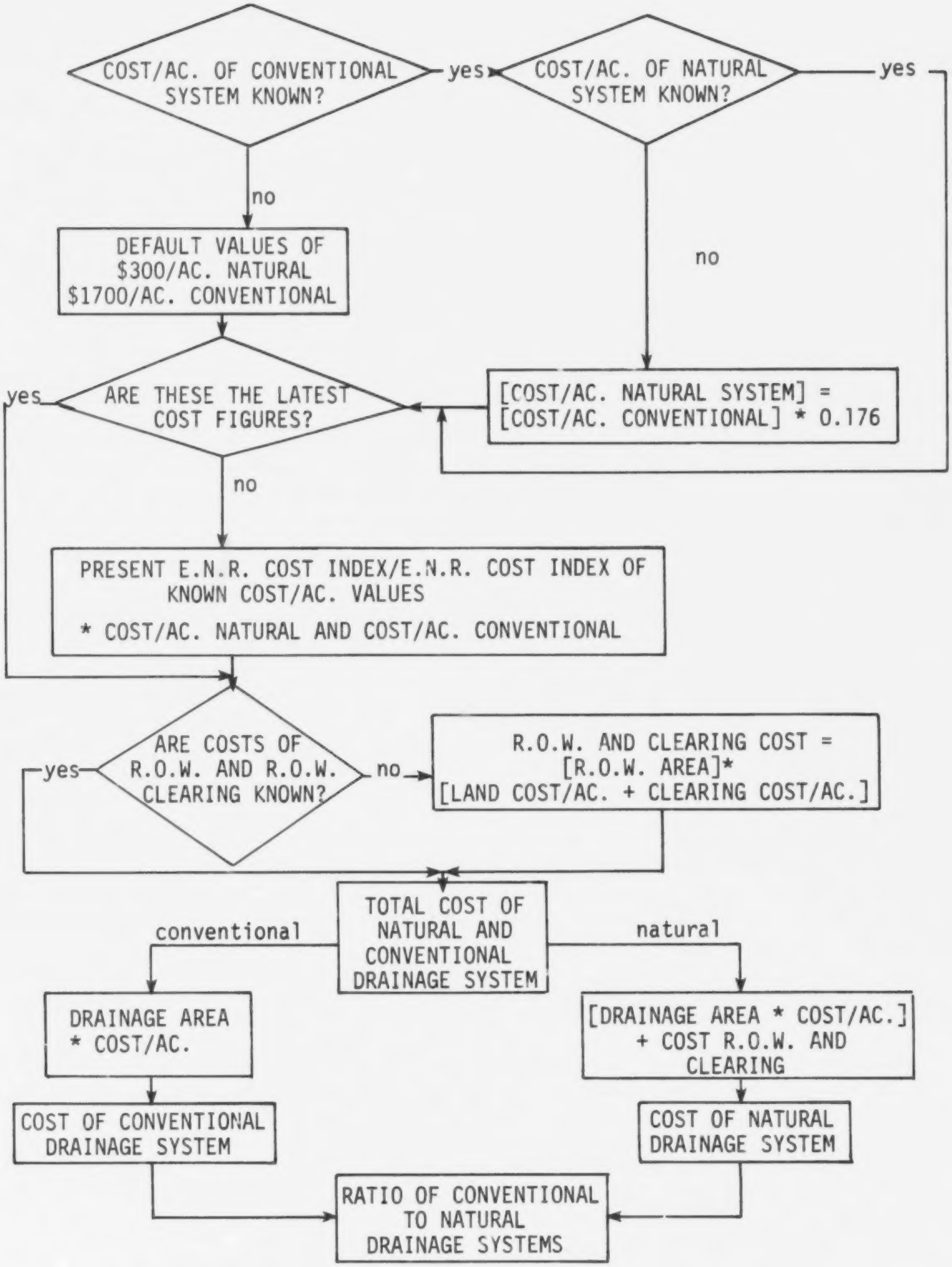

COST/AC. OF CONVENTIONAL SYSTEM KNOWN?
yes
COST/AC. OF NATURAL SYSTEM KNOWN?
yes
no
DEFAULT VALUES OF $300/AC. NATURAL $1700/AC. CONVENTIONAL
no
yes
ARE THESE THE LATEST COST FIGURES?
[COST/AC. NATURAL SYSTEM] = [COST/AC. CONVENTIONAL] * 0.176
no
PRESENT E.N.R. COST INDEX/E.N.R. COST INDEX OF KNOWN COST/AC. VALUES
* COST/AC. NATURAL AND COST/AC. CONVENTIONAL
yes
ARE COSTS OF R.O.W. AND R.O.W. CLEARING KNOWN?
no
R.O.W. AND CLEARING COST = [R.O.W. AREA]* [LAND COST/AC. + CLEARING COST/AC.]
TOTAL COST OF NATURAL AND CONVENTIONAL DRAINAGE SYSTEM
conventional
natural
DRAINAGE AREA * COST/AC.
[DRAINAGE AREA * COST/AC.] + COST R.O.W. AND CLEARING
COST OF CONVENTIONAL DRAINAGE SYSTEM
COST OF NATURAL DRAINAGE SYSTEM
RATIO OF CONVENTIONAL TO NATURAL DRAINAGE SYSTEMS

Fig. 12 Subroutine CSTANL system logic

3. If total right-of-way acquisition and clearing costs are not available, they are computed by use of unit costs for land acquisition and clearing.

4. The total costs of conventional and natural drainage systems for each area are computed and the ratio of one to the other indicates the relative efficiencies of each drainage system.

The results from Subroutine CSTANL are subject to the accuracy of the input cost data.

AREA-DISCHARGE DATA FOR NATURAL SECTIONS

The SWMM uses normalized area-discharge curves for flow routing through symetrical cross-sections in the Transport Block (9). These curves are programmed into the SWMM through Block Data for thirteen element types such as circular, rectangular and trapezoidal sections (2). Also there are two user supplied element types for modeling natural stream or other nonsymetrical cross-sections. The normalized area-discharge curves for these sections must be manually calculated, as shown in Figure 13, and input to the SWMM. When modeling a natural drainage system the user will often find that the flow is not confined within the natural channel. Also the overbanks of a natural cross-section will generally have Manning's roughness coefficient, n, values significantly different from that of the channel. Therefore, calculation of an area-discharge curve for a natural cross-section must consider the complex relationships between depth of flow and flow area and between depth of flow and a composite Manning's n value. Following project objectives, subroutine NATSEC was developed to generate normalized area-discharge curves for irregular or nonsymetrical cross-sections for which Manning's n varies with stage. The cross-section, as shown in Figure 14, is described by a two dimensional linear coordinate system and three Manning's n values, one n value for each overbank and one for the channel.

When the depth of flow is within the channel the flow calculation is a direct application of Manning's uniform flow equation. If the flow depth is such that flow is not contained within the channel, Manning's equation is applied separately to each section as shown in Figure 14. The total discharge is assumed to be the sum of individual discharges. This assumption was used by Chow (10) in his derivation of an equation for an equivalent roughness in a channel with composite roughness.

Input to subroutine NATSEC consists of a set of progressively increasing coordinate points and the desired number of area increments. A maximum of 25 coordinate points and 50 area increments are allowed. Also the horizontal stationing for the left and right overbanks is required.

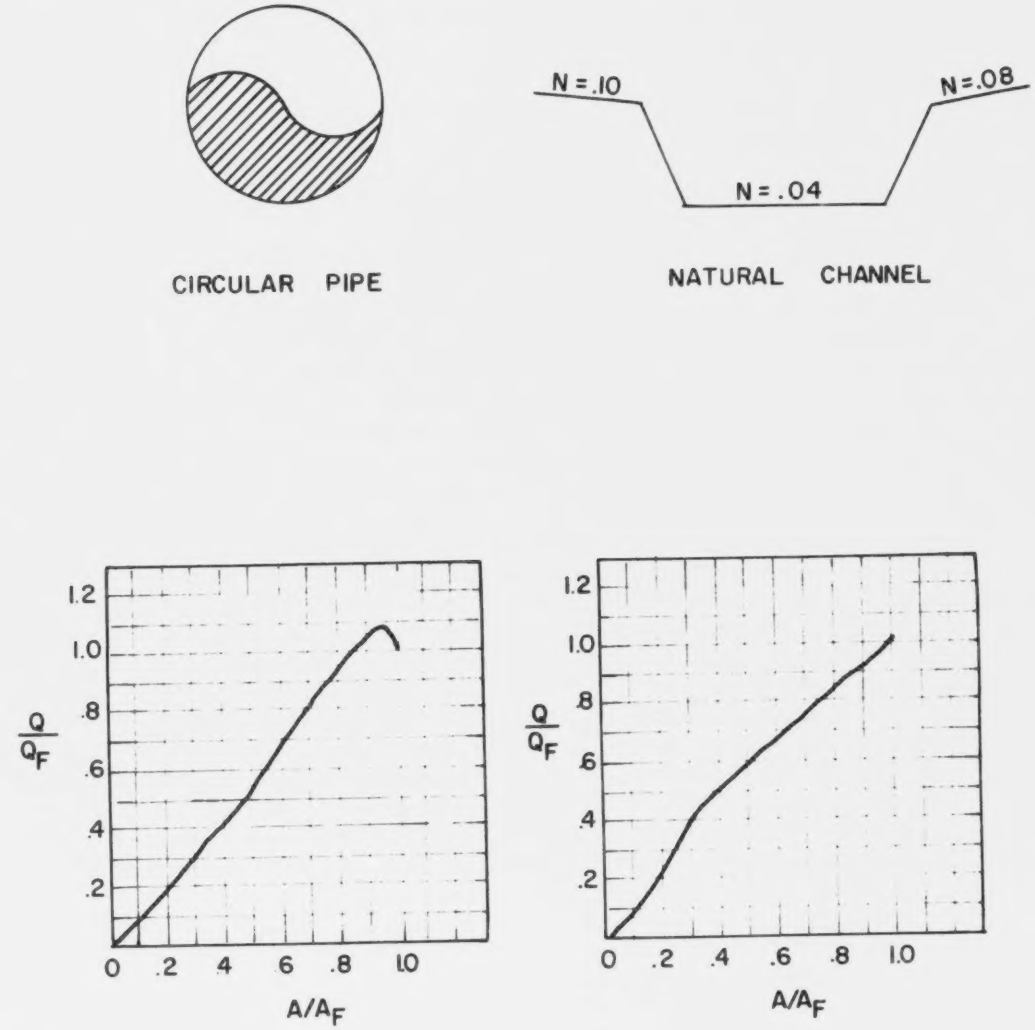


Fig. 13 Normalized area-discharge curves

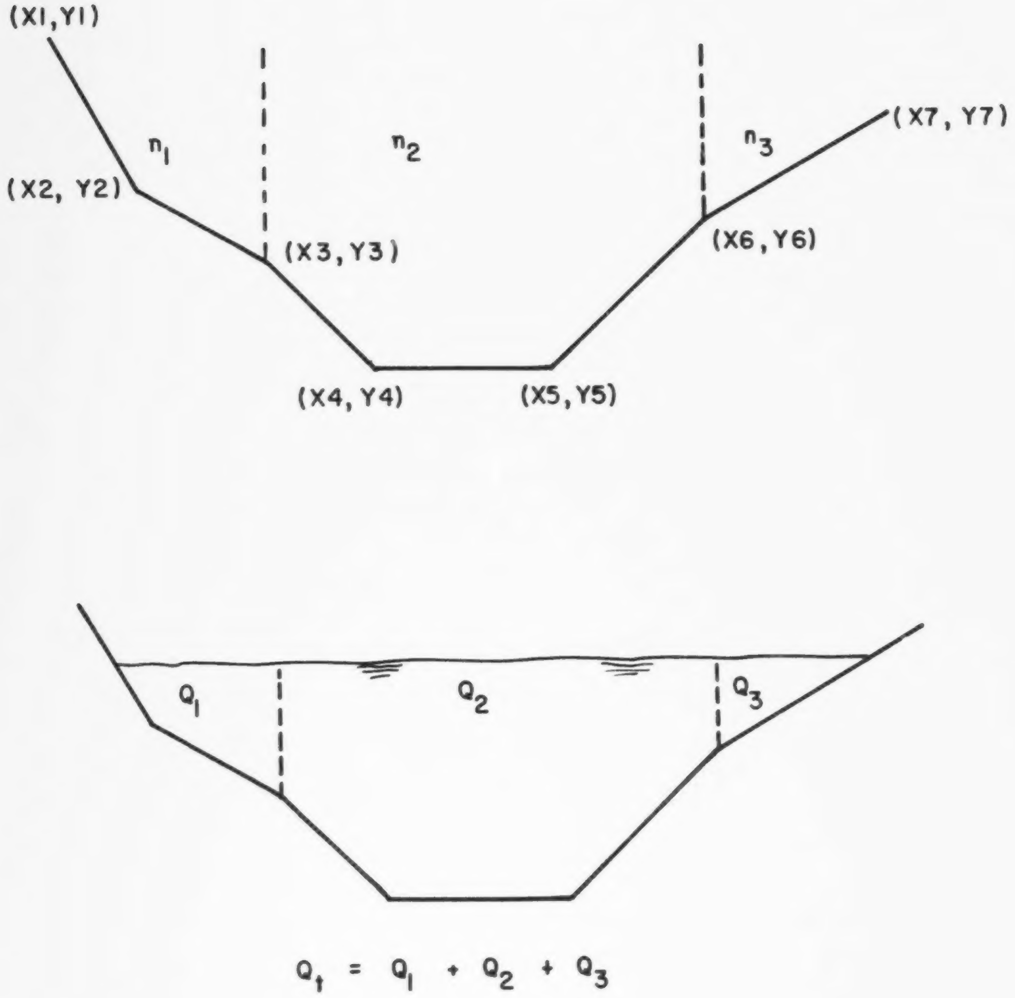

(X1,Y1)
(X2, Y2)
(X3, Y3)
(X4, Y4)
(X5, Y5)
(X6, Y6)
(X7, Y7)
n_1
n_2
n_3
Q_1
Q_2
Q_3
$Q_t = Q_1 + Q_2 + Q_3$

Fig. 14 Modeling of natural cross-sections

The storage arrays have been expanded to allow the storage of normalized area-discharge data for ten natural cross-sections. Further versatility is provided by allowing the user to geometrically enlarge or reduce each cross-section by use of an input proportional constant.

The subroutine calculates the full flow area by dividing the cross-section into the triangles, rectangles or trapezoids defined by the input coordinate system and the maximum water surface elevation. The maximum water surface elevation is taken to be the lower elevation (lower Y value) corresponding to the two end points of the cross-section (lowest or highest X value). The areas of these uniform geometrical shapes are then calculated and summed to give the full flow area. Each incremental area is determined by dividing the full flow area by the user specified number in increments. The subroutine subtracts the area increment from the full flow area and uses an iteration process to calculate the depth of flow corresponding to the new flow area. With the depth of flow defined for an incremental area the subroutine calculates the corresponding hydraulic radius from the known geometry.

To define the relationship between depth of flow and an equivalent roughness value the equation developed by Lotter (11) for a composite cross-section is used. Lotter assumed that the total discharge is equal to the sum of the discharge of the subdivided areas. Thus, the equivalent roughness is:

$$n_e = \frac{PR^{1.67}}{\sum_{i=1}^{s} \frac{P_i \quad R_i^{1.67}}{n_i}}$$

n_e = equivalent roughness

P = total wetted perimeter

P_i = wetted perimeter of subsection

R = hydraulic radius of total section

R_i = hydraulic radius of subsection

n_i = roughness of subsection

s = total number of subsections

Subroutine NATSEC allows a natural cross-section to be divided into three subsections, two overbank sections and a channel section with Manning's n value input for each section.

Using the calculated depth-area relationship and depth-roughness relationship the subroutine calculates the normalized flow-area curves as follows:

$$\frac{Q_s}{Q_f} = \frac{\dfrac{A_s\, R_s^{0.67}}{n_s}}{\dfrac{A_f\, R_f^{0.67}}{n_f}}$$

subscript s refers to a subsection
subscript f refers to a full section
Q = discharge
A = area
R = hydraulic radius
n = roughness coefficient

In the above equation the slope terms cancel out because channel and overbank slopes are assumed to be equal. The output from subroutine NATSEC is a tabular version of the normalized area-discharge curves in Block Data of the SWMM. Subroutine NATSEC was used to great advantage in modeling runoff from The Woodlands where all of the channel system described to the SWMM consists of natural channels.

MODELING OF POROUS PAVEMENT

The earliest applications of porous pavements were for nonstorage purposes; they were used on top of a regular pavement to provide improved drainage and reduce the possibility of skidding or hydroplaning. The use of porous pavements as stormwater management systems was initiated by Franklin Institute in Philadelphia, Pennsylvania under sponsorship of the U. S. Environmental Protection Agency(12).

The primary benefit derived from porous pavements is an appreciable reduction of runoff rate and volume from impervious urban areas. If the pavement and base are designed adequately, all of the runoff may be captured, detained and released at a slower rate to prevent increases in flood flows. Concurrently, the stored water may be allowed to infiltrate into the natural ground.

Porous pavements may also be used in areas that are already urbanized such as downtown areas of most cities as well as existing shopping centers where the storm sewer network was installed prior to excessive impervious cover development. Under these conditions, the storm sewers may become overloaded and if parking lot or roof storage is not a design criterion, the disposal of excess runoff becomes a problem that porous pavements could solve. This benefit is enhanced in areas with combined sewerage because the probability of sewer overloading and the resultant discharge of raw sewage into the receiving waters is reduced.

In areas of slight topography or with minimal soil depths, the cost of installing storm sewers is very high because both sewer size and excavation volumes are high. The use of porous pavements in these areas reduces both sewer size and excavation depth, thus resulting in a net savings in drainage costs.

If the stormwater requires treatment, it may be stored in porous pavement systems isolated from the natural ground by an impermeable membrane until the treatment plant capacity becomes available. Thus, treatment plant capacity does not need to be expanded. Also detention of highly polluted initial runoff by the porous pavement and dilution by less polluted subsequent runoff can result in reduced pollutant concentrations throughout the storm.

Because impermeable pavements preclude the survival of any vegetation, natural vegetation and drainage patterns can be retained by the use of porous pavements. Consequently, the clearing of large areas for parking lots is unnecessary and secondary aesthetic benefits are also derived. Other benefits in the form of construction cost reductions, elimination of curbs and gutters, enhanced water supply, traffic safety resulting from skid resistance and improved visibility on wet pavements, and possible use of solid wastes for base material are discussed by Thelen et al. (12).

Development of Model and Theory

An extensive modeling effort was undertaken to develop a comprehensive analysis of flow and storage in porous pavement systems and thereby define the environmental effects of this type of pavement. The mechanics of flow through a porous pavement system has not received much attention; and only after a complete understanding of porous pavement operations will the total applicability of this system be determined.

A deliberate attempt was made to keep the model as simple as possible and yet to provide adequate quantification of the hydrologic responses of a porous pavement. Also, the effects of a variety of different pavement characteristics and configurations can be evaluated. This will allow for the investigation of various porous pavement systems to determine the optimum system especially during planning phases of the project.

The hydrologic responses of a porous pavement may be simulated by a system of hydraulically connected control volumes for which the inflows and outflows are mathematically defined. The porous pavement, the subgrade and the natural ground (or the drain system) are considered to be sequential but internally independent storage reservoirs.

The basic equation of continuity or conservation of mass is applied to each reservoir:

$$\frac{ds}{dt} = I - 0$$

where

I = inflow into the reservoir
0 = outflow from the reservoir
$\frac{ds}{dt}$ = change in storage volume

As shown in Figure 15, the porous pavement area would serve to control runoff from contributing impervious areas. Therefore, inflow to the porous pavement system, RUNOFF is defined as:

$$\text{RUNOFF} = \text{PAV} + \text{HYD}$$

where

PAV = direct rainfall onto the porous pavement
HYD = surface runoff hydrograph from contributing areas

Contributing areas to the porous pavement will generally be developed and impervious in nature. Consequently, the surface runoff hydrograph from contributing areas is determined by use of the method developed by Izzard (13). This method, selected for its programming ease, utilizes a dimensionless hydrograph from paved areas as shown in Figure 16. The key parameters in this method are time to equilibrium, t_e; equilibrium flow, q_e; equilibrium surface detention volume, V_e; the intensity of rainfall, i; and the length of overland flow, L. The following equations define these parameters:

$$q_e = \frac{iL}{43200}$$

where: q_e is in cfs, i in in/hr, and L in ft.

$$V_e = \frac{k\ LY^{4/3}\ i^{1/3}}{3.51}$$

where: k is an empirically derived lumped coefficient for the effects of slope and flow retardance of the pavement, Y is the flow depth.

$$t_e = \frac{V_e}{30\ q_e}$$

Using t/t_e values based on the computation interval and Figure 16, the q/q_e values and the corresponding q values are determined for the rising limb of the hydrograph. The β factor, defined as:

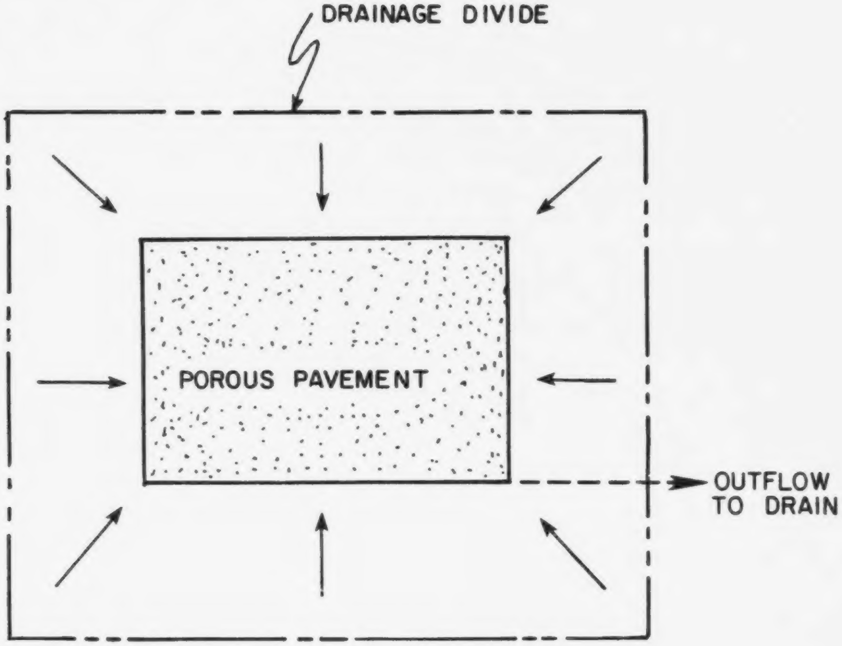
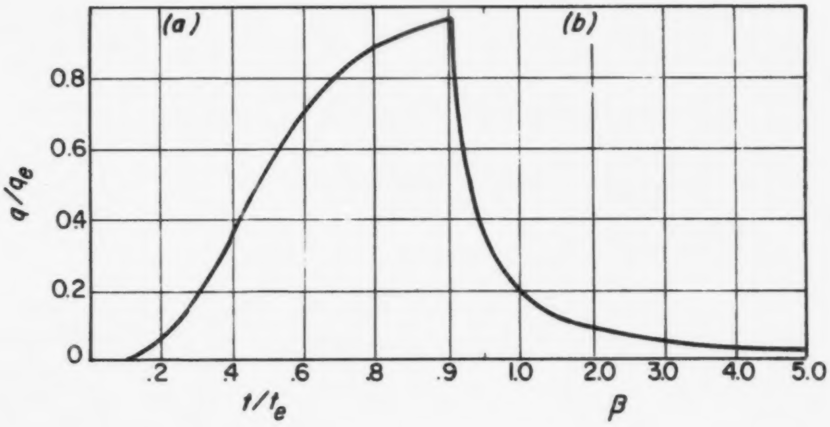

DRAINAGE DIVIDE
POROUS PAVEMENT
OUTFLOW TO DRAIN

Fig. 15 Porous pavement and surrounding drainage area



From: Linsley, Kohler & Paulhus, 1975

Fig. 16 Izzards dimensionless hydrograph for overland flow

$$\beta = \frac{60\ q_e\ t_a}{V_o}$$

where: t_a = time after rainfall

V_o = equivalent to V_e without the rainfall intensity component

is used to determine the q/q_e and corresponding q values for the recession limb of the hydrograph. The dimensionless hydrograph in Figure 16 is represented by the following equations in the model:

$$0 < t/t_e \leq 0.15 \qquad q/q_e = e^A$$

where: $A = 32.2 \ln (t/t_e) - 8.36$

$$0.15 < t/t_e \leq 0.35 \qquad q/q_e = e^B$$

where: $B = 11.64 \ln (t/t_e) - 5.203$

$$0.35 < t/t_e \leq 0.55 \qquad q/q_e = 1.992(t/t_e) - 0.432$$

$$0.55 < t/t_e \leq 0.75 \qquad q/q_e = \ln C$$

where: $C = 0.533 + 2.46\ e^T$
$T = t/t_e$

$$0.75 < t/t_e \leq 1.0 \qquad q/q_e = \ln D$$

where: $D = 1.628 + 1.023\ e^T$

$T = t/t_e$

$$t/t_e > 1.0 \qquad q/q_e = (2.0\ \beta + 1.0)^{-3/2}$$

The rainfall hyetograph is input as average intensity per computation interval for all intervals during which rainfall occurs. Runoff hydrographs are computed for each interval, successively, and summed to determine the cumulative storm hydrograph from each paved area. The cumulative hydrographs from all paved areas are added to obtain the total storm hydrograph from contributory areas to the porous pavement. The inflow hydrograph is converted to units of depth based on the area of the porous pavement and the computation interval. The rainfall depth onto the porous pavement, PAV, is added to surface runoff depth, HYD, to determine RUNOFF. An alternative user optional input table of RUNOFF depths per computation interval may also be utilized. This option is useful in those instances where storm hydrograph data are available from observation or computed by other methods.

As considered in this model, the outflows from the porous pavement system are composed of four outflow functions defined as follows:

$$0_{total} = 0_{vert} + 0_{hor} + 0_{surf} + 0_{evap}$$

0_{vert} is the vertical seepage into the pavement, base, or ground. This seepage is determined as the difference in surface water depth at the beginning and end of each time interval. The variable head permeability equation as defined by Taylor (14) is

$$K = 2.3 \frac{a\ L}{A\Delta t} \log \frac{h_1}{h_2}$$

where

K = permeability of flow element
a = cross-sectional area of surface water
A = cross-sectional area of flow element
L = thickness of flow element
h_1 = depth of surface water at time t
h_2 = depth of surface water at time t = t, + t

In a porous pavement system the cross-sectional areas of surface water and flow element are always equal and so the equation is reduced to

$$K = 2.3 \frac{L}{\Delta t} \log \frac{h_1}{h_2}$$

This equation may be arranged to solve for h as follows:

$$h_2 = \frac{h_1}{10^E}$$

where $E = \frac{K\Delta t}{2.3L}$

Then, vertical seepage is equal to the change in water depth during Δt or,

$$0_{vert} = h_1 - h_2$$

0_{hor} is the lateral outflow to a drain or into the natural ground as a result of water storage in the base and pavement. This condition is analagous to bank recharge from a rising stream and for homogenous isotropic aquifers of finite width, the influence of each increment of rise in the stream is determined by the following set of equations (15):

$$\frac{d^2h}{dx^2} = \frac{S}{T}\frac{dh}{dt}$$

$$h(o,t) = 0 \text{ for } t \leq 0$$

$$h(o,t) = H_i \text{ for } t > 0$$

$$\frac{dh(L,t)}{dx} = 0$$

$$h(x,o) = 0$$

where

h = hydraulic head or water depth
x = distance from boundary
S = coefficient of storage of aquifer
T = aquifer transmissivity
H_i = change in water depth at boundary = 0_{vert}
L = total width of aquifer

Integrate the first equation and apply boundary conditions to determine that the constant of integration is equal to zero and the result is:

$$\frac{dh}{dx} = \frac{S}{T}\frac{dh}{dt}x$$

The Darcy flow equation can be extended by continuity to define net flow rate as follows:

$$V = K\frac{dh}{dx}$$

$$Q = AV = K A\frac{dh}{dx} = K h w\frac{dh}{dx}$$

$$q = Q/W = K h\frac{dh}{dx} = T\frac{dh}{dx}$$

where

V = velocity of flow
K = permeability of flow element
$\frac{dh}{dx}$ = change in hydraulic grade
Q = total mass flow rate
A = total flow area
h = depth of flow
w = width of flow
q = flow rate per unit width
T = transmissivity of flow element

Substitute

$$q = \frac{T}{T} \frac{S}{T} \frac{\partial h}{\partial t} x = \frac{S \partial h}{\partial t} x$$

define

$$\frac{\partial h}{\partial t} = \frac{h_1 - h_2}{\Delta t}$$

and S is the storage coefficient of the natural ground. Then at a distance x from the porous pavement side, the discharge per unit width is defined as:

$$q = S\left(\frac{h_1-h_2}{\Delta t}\right) x$$

Because the volume of flow remaining in the porous pavement system is the only item of interest, the value of x was arbitrarily set equal to 1.0.

Then lateral outflow = $q \; P \; \Delta t = S\left(\frac{h_1-h_2}{\Delta t}\right) P \; \Delta t$

where P is the pavement perimeter. 0_{surf} is the surface runoff resulting from ponding on top of the porous pavement, which occurs either because the inflow rate is greater than the porous pavement permeability or the total storage capacity in the porous pavement system is exceeded. The model requires a depth-storage relationship to determine when the storage is exceeded. On a horizontal pavement, the model determines the depth-storage relationship by use of input pavement and subgrade depths and porosities; on a sloping pavement, this relationship has to be independently computed and input to the model.

The surface runoff from a horizontal pavement is defined by the weir equation:

$$Q = CLH^{3/2} \text{ where}$$

C = input weir coefficient
L = input weir length
H = $h - h_o$
h_o = dead surface storage on the porous pavement
h = depth of flow on the porous pavement

On a sloping porous pavement Manning's Equation is used to determine the surface runoff.

$$Q = yL \, \frac{1.486}{n} \, y^{2/3} \, S^{1/2}$$

where

y = computed depth of flow
L = input width of flow
n = input roughness coefficient

s = input energy slope

0_{evap} is the volume of water lost to evaporation. Either monthly, weekly or daily evaporation rates may be input to the model; the monthly and weekly rates are divided into average daily rates. The daily evaporation rate is increased by 25% to allow for heat absorption by the dark asphalt. The model only allows for evaporation from 6 a.m. to 8 p.m. with the maximum rate at 2 p.m. As shown in Figure 17 a triangular distribution of evaporation is developed by the model by use of the equation:

$$E_p = \frac{E_t}{7}$$

where

E_p = peak eveporation rate, inches/hr

E_t = total daily evaporation, inches

for $0 < t_c \leq 6$ $\quad E = 0$

for $6 < t_c \leq 14$ $\quad E = E_p(\frac{t_c - 6}{8})$

for $14 < t_c \leq 20$ $\quad E = E_p(\frac{20 - t_c}{6})$

for $20 < t_c \leq 24$ $\quad E = 0$

where

t_c = clock time in hours

E = instantaneous evaporation rate

Model Operation

The paths of water flow through the porous pavement system are shown in Figure 18. For each computational time interval, all inflows and outflows are accounted for. The total runoff hydrograph, in inches per computational time interval, is either input to the model or may be computed as the sum of the runoff hydrograph from contributory areas and direct rainfall onto the pavement as described previously.

The following sequential computational steps as illustrated in the flow chart, Figure 19, are then performed:

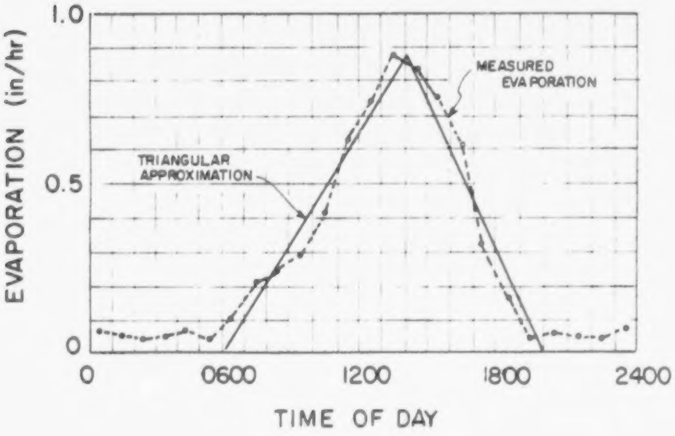
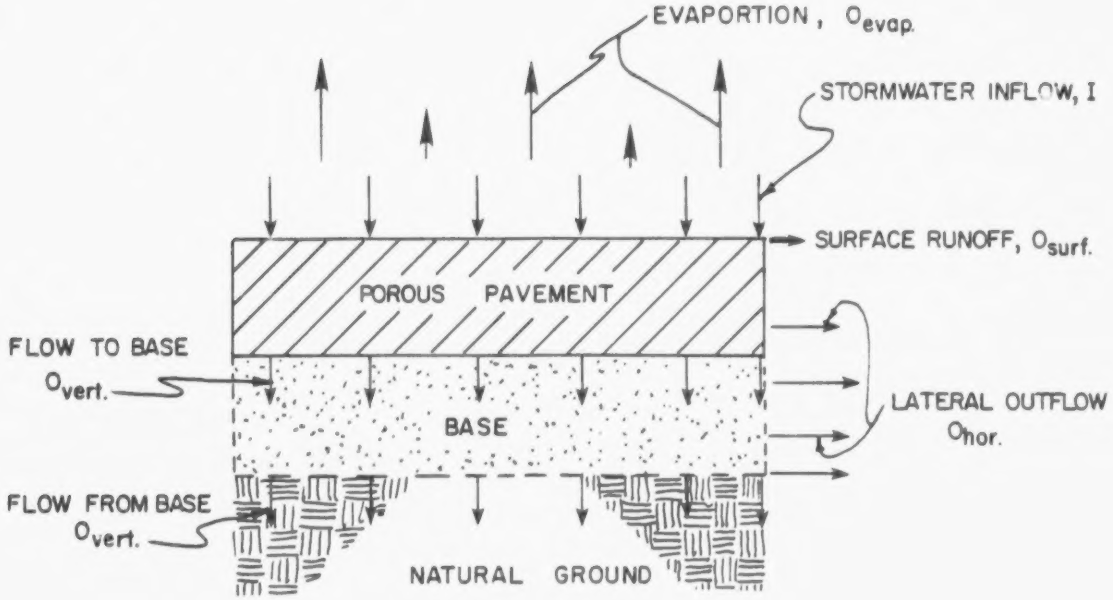


Fig. 17 Triangular approximation of evaporation



EVAPORTION , $O_{evap.}$
STORMWATER INFLOW, I
SURFACE RUNOFF, $O_{surf.}$
POROUS PAVEMENT
FLOW TO BASE $O_{vert.}$
BASE
LATERAL OUTFLOW $O_{hor.}$
FLOW FROM BASE $O_{vert.}$
NATURAL GROUND

Fig. 18 Pavement cross-section and modeled flow

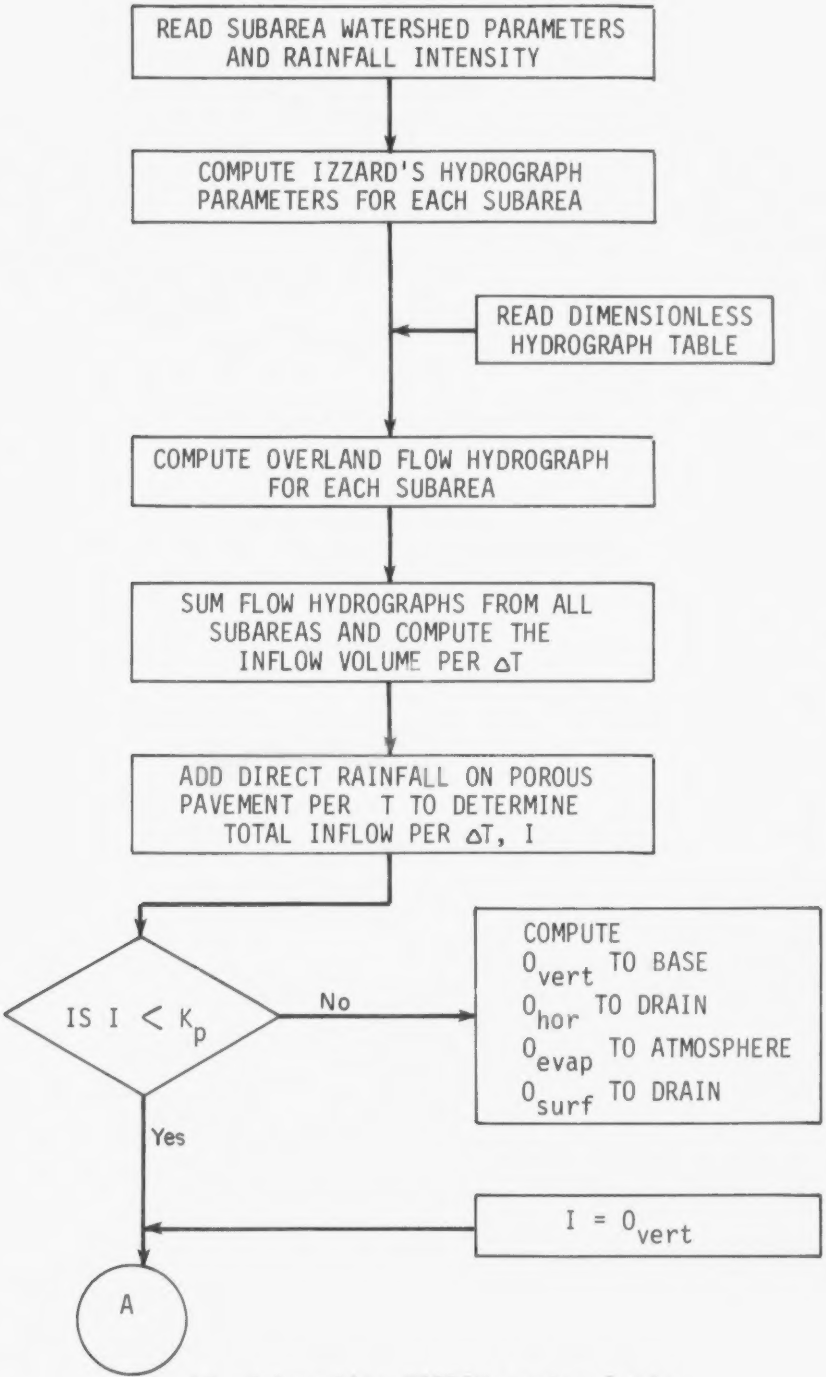

READ SUBAREA WATERSHED PARAMETERS AND RAINFALL INTENSITY
COMPUTE IZZARD'S HYDROGRAPH PARAMETERS FOR EACH SUBAREA
READ DIMENSIONLESS HYDROGRAPH TABLE
COMPUTE OVERLAND FLOW HYDROGRAPH FOR EACH SUBAREA
SUM FLOW HYDROGRAPHS FROM ALL SUBAREAS AND COMPUTE THE INFLOW VOLUME PER ΔT
ADD DIRECT RAINFALL ON POROUS PAVEMENT PER T TO DETERMINE TOTAL INFLOW PER ΔT, I
IS I < K_p
No
Yes
COMPUTE
O_vert TO BASE
O_hor TO DRAIN
O_evap TO ATMOSPHERE
O_surf TO DRAIN
I = O_vert
A

Fig. 19 Subroutine PORPAV system logic

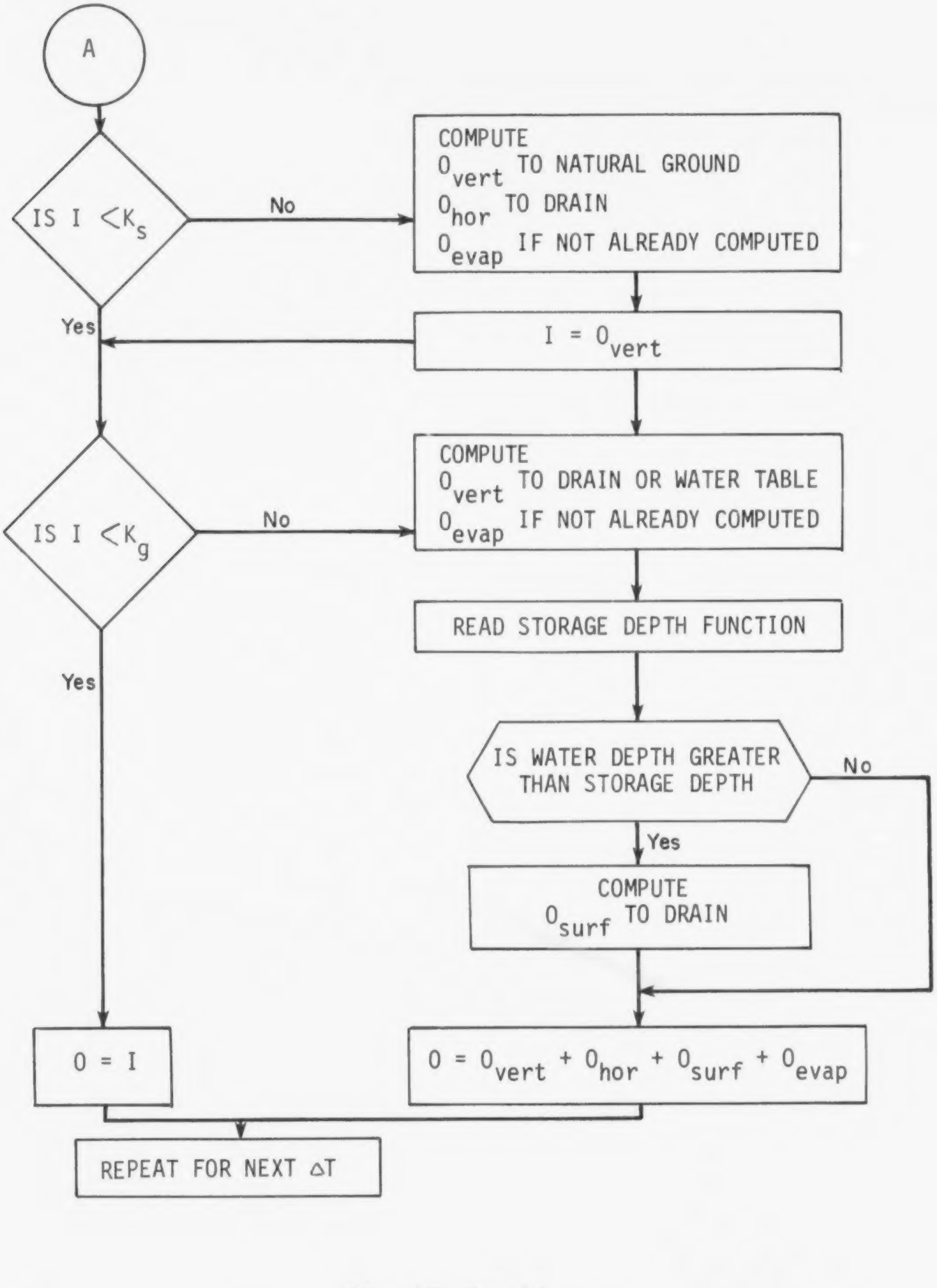

A
IS I < K_s
No
COMPUTE
O_{vert} TO NATURAL GROUND
O_{hor} TO DRAIN
O_{evap} IF NOT ALREADY COMPUTED
Yes
I = O_{vert}
IS I < K_g
No
COMPUTE
O_{vert} TO DRAIN OR WATER TABLE
O_{evap} IF NOT ALREADY COMPUTED
READ STORAGE DEPTH FUNCTION
Yes
IS WATER DEPTH GREATER THAN STORAGE DEPTH
No
Yes
COMPUTE
O_{surf} TO DRAIN
O = I
O = O_{vert} + O_{hor} + O_{surf} + O_{evap}
REPEAT FOR NEXT ΔT

Fig. 19 Cont'd

1. Evaporation losses in inches per computational time interval are computed and subtracted from the sum of the runoff depth and previous surface storage, if any.

2. The volume of runoff after allowing for infiltration is compared to the permeability in inches per time interval of the porous pavement. As in most cases the permeability is much greater than the inflow runoff rate and all of the water moves into the pavement control volume. In those cases where the permeability has been severely reduced and is less than the inflow rate, the inflow into the pavement is computed as the vertical seepage into the pavement and the excess is stored on the surface of the pavement for later computation of surface runoff from the pavement.

3. The inflow into the pavement control volume is added to the storage volume in the pavement and then compared to the permeability, in inches per computational time interval, of the base. If the base permeability is greater than the inflow into the pavement, then all of the flow is transferred into the base control volume. This is true for most porous pavement systems operating according to design. In those instances when the base permeability is less than the inflow volume, the inflow into the base is computed as the vertical seepage into the base. The lateral outflow from the pavement is also computed if an impermeable membrane is not installed along the pavement perimeter. The difference between the inflow into the pavement and the outflows (vertical and lateral) from the pavement is stored in the pavement.

4. The inflow into the base control volume is added to the storage volume in the base and then compared to the permeability in inches per computational time interval, of the natural ground. If the bottom is sealed with an impermeable membrane, then the permeability is set equal to zero, and no flow is lost to the natural ground. The flow volume remaining in the base after vertical seepage into the natural ground, is compared to the drain capacity, in inches per computational time interval. If the natural ground permeability and/or drain capacity are inadequate to remove all of the flow in the base, the vertical seepage into the natural ground and drains, as well as the lateral outflow, if any, is computed. The difference between the inflow into the base and the outflows (vertical and lateral) from the base is stored in the subgrade.

5. All stored volumes are compared to available volumes.

> If storage volume in the base is exceeded, the excess is stored in the pavement; if storage volume in the pavement is exceeded, the excess is added to the surface storage on the pavement, if any exists. Surface runoff is then computed either as broad channel flow or weir flow from the pavement to an adjacent drainageway.

After continuity conditions are satisfied by comparing all inflows to outflows, this computational procedure is repeated for each time interval in the inflow hydrograph. The surface and drain outflows are stored in retrievable arrays. The primary output objective at present is the surface runoff, if any. But the other output variables allow for a thorough examination of the hydraulic operational characteristics of the porous pavement system, including the analysis of the desirability or adequacy of the drains and the discharge rate from the drains.

Model Application

Subroutine PORPAV, as the porous pavement model is called, has been applied to the porous pavement parking area in The Woodlands. All existing data for this porous pavement area is not sufficiently comprehensive to test all of the model capabilities. No record of drain capacity is available. Also, the available data on storage of runoff in the porous pavement system are in terms of percent of depth that the probe is submerged. These data have to be converted to inches of depth of water storage to make the data useful. If the probe were exactly as long as the base thickness then the conversion would be relatively simple. But such is not the case; the probes have to be calibrated. Consequently these data could not be used to verify and calibrate subroutine PORPAV. A further problem arises from the fact that due to the size of the observation wells (6 inches diameter), whenever a water sample is withdrawn a significant drop in water elevation in the well is observed. Therefore, comparisons of observed and simulated events were limited to general tests, e.g., if surface runoff was observed. Pavement and base permeabilities had to be reduced and a design storm applied as shown in Table 3 in order to generate surface runoff. Contributary area boundaries were also enlarged so that the hydrograph prediction capabilities of the model could be hypothetically tested.

The porous pavement installation at The Woodlands is constructed as a layer of open-graded asphalt concrete underlain by a gravel base course with appreciable storage capacity. The whole system is isolated from the natural ground by an impermeable polyethylene liner. Water is removed from the base by an artificial drain. As shown in Figure 20 the pavement area is rectangular, approximately 64 meters (210 feet) by 39.6 meters (130 feet) in size. Two contributory areas are identified, 1865.38 sq. meters (6120 sq ft) and 1828.8 sq. meters (6000 sq ft

TABLE 3. INPUT DATA TO POROUS PAVEMENT MODEL

```
* * * POROUS PAVEMENT AT THE WOODLANDS * * * DESIGN STORM* * *


NUMBER OF CONTRIBUTORY SUBAREAS =          2
NUMBER OF RAINFALL TIME INTERVALS =        7
NUMBER OF COMPUTATIONAL STEPS =          150
NUMBER OF RUNOFF TIME INTERVALS =         -0*


     SUBAREA CHARACTERISTICS

SUBAREA        ROUGHNESS      SLOPE ft/ft  LENGTH, ft    WIDTH, ft

   1              .050          .030         34.0          160,0
   2              .017          .010         80.0           75,0


             INPUT RAINFALL DATA, in/hr

1,500000  3,000000  4,500000  6,000000  4,500000  3,000000  1,500000

 COMPUTATIONAL TIME INTERVAL      2,000  MINUTES


             INPUT EVAPORATION DATA

 STARTING TIME  12,HR  30,MIN

 EVAPORATION DEPTH FOR   30 DAYS IS    5,000 INCHES


             POROUS PAVEMENT CHARACTERISTICS

                         PERM.     DEPTH     POROS-      INIT.      MAX.
                         COEFF      in       ITY in/hr   STOR.      STOR.

 PAVEMENT               15.000     2.500     30.000     -0,000       ,750

 SUBGRADE               30.000    12.000     50.000     -0.000      6,000

 NATURAL GROUND           .500    36.000      5.000     -0.000      1,800

 ARTIFICIAL DRAIN        6.000     3,000


     WIDTH OF PAVEMENT =                                  130,000
     LENGTH OF PAVEMENT =                                 210,000
     SLOPE OF PAVEMENT =                                     ,030
     DOWNSTREAM FLOW WIDTH OF PAVEMENT =                   15,000
     MANNING COEFFICIENT FOR PAVEMENT =                      ,050
     WEIR COEFFICIENT FOR PAVEMENT =                       -0,000
     DEAD STORAGE ON PAVEMENT =                              ,500
     INITIAL STORAGE ON PAVEMENT =                         -0,000
```

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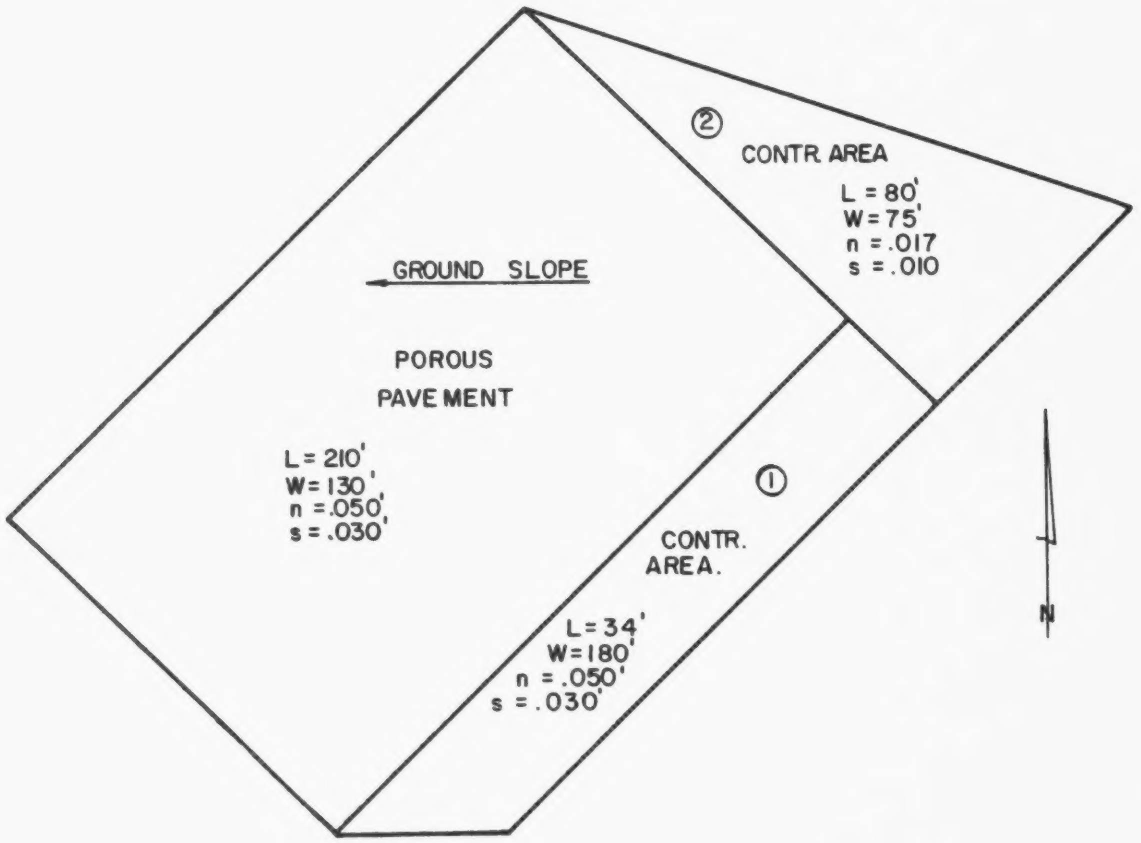

② CONTR. AREA
L = 80'
W = 75'
n = .017
s = .010
GROUND SLOPE
POROUS
PAVEMENT
L = 210'
W = 130'
n = .050'
s = .030'
① CONTR.
AREA.
L = 34'
W = 180'
n = .050'
s = .030'
N

Fig. 20 Porous pavement test area

in area, respectively.

Based on general criteria, initial testing of the model indicated that storage and outflow were adequately simulated for observed events. The design input rainfall hyetograph and corresponding runoff hydrograph from each contributory area are shown in Figure 21. Flow and storage in each control volume as predicted by the model for two different permeabilities in each pavement element are depicted in Figures 22 and 23. Figure 22 illustrates the porous pavement system operation when pavement and base permeabilities are 101.6 cm/hr (40 in/hr) and 203.2 cm/hr (80 in/hr), respectively. As shown in Figure 22, the design storm used (based on 100 year rainfall for the Houston area) did not generate any surface runoff because of the high infiltration rates available. Therefore, the permeabilities were reduced to 38.1 cm/hr (15 in/hr) and 76.2 cm/hr (30 in/hr) for the pavement and base, respectively. The resulting model predictions include surface runoff as shown in Figure 23. Table 4 shows a sample segment of the model output.

The environmental effects of porous pavement could not be determined due to the lack of sufficient data. However, specific trends were evident. Runoff which accumulated in the porous pavement system exhibited significant (on the order of magnitude of four) nitrate, nitrite, and Kjeldahl nitrogen concentrations in comparison to surface runoff. Water stored in the sand subbase was high in orthophosphates and total phosphates. Soluble COD concentrations seemed to be much lower in water stored within the porous pavement system. Sample average water quality is illustrated for the storm of 20 February 1976 in Figures 24 and 25. Suspended solids data were not compiled and consequently, the effect of this parameter is unknown.

The fate of pollutants stored in the porous pavement system could not be determined because no data were available for water quality during periods between storm events. Generally, the water would have drained out of the porous pavement system within a short time after a storm event. The drain water also was not sampled so that the effect of runoff retardation within the porous pavement system could not be determined.

Subroutine PORPAV at present only allows for dilution of pollutant concentrations by retarding runoff. In general the drain from a porous pavement system will discharge into a receiving body of water specifically designed for this purpose, e.g., channel, pond or wastewater treatment plant. Therefore, surface runoff would be the only consequential flow with regard to downstream runoff quantity and quality. Subroutine PORPAV will determine outflow hydrographs from both the drain as well as surface runoff as shown in Table 5.

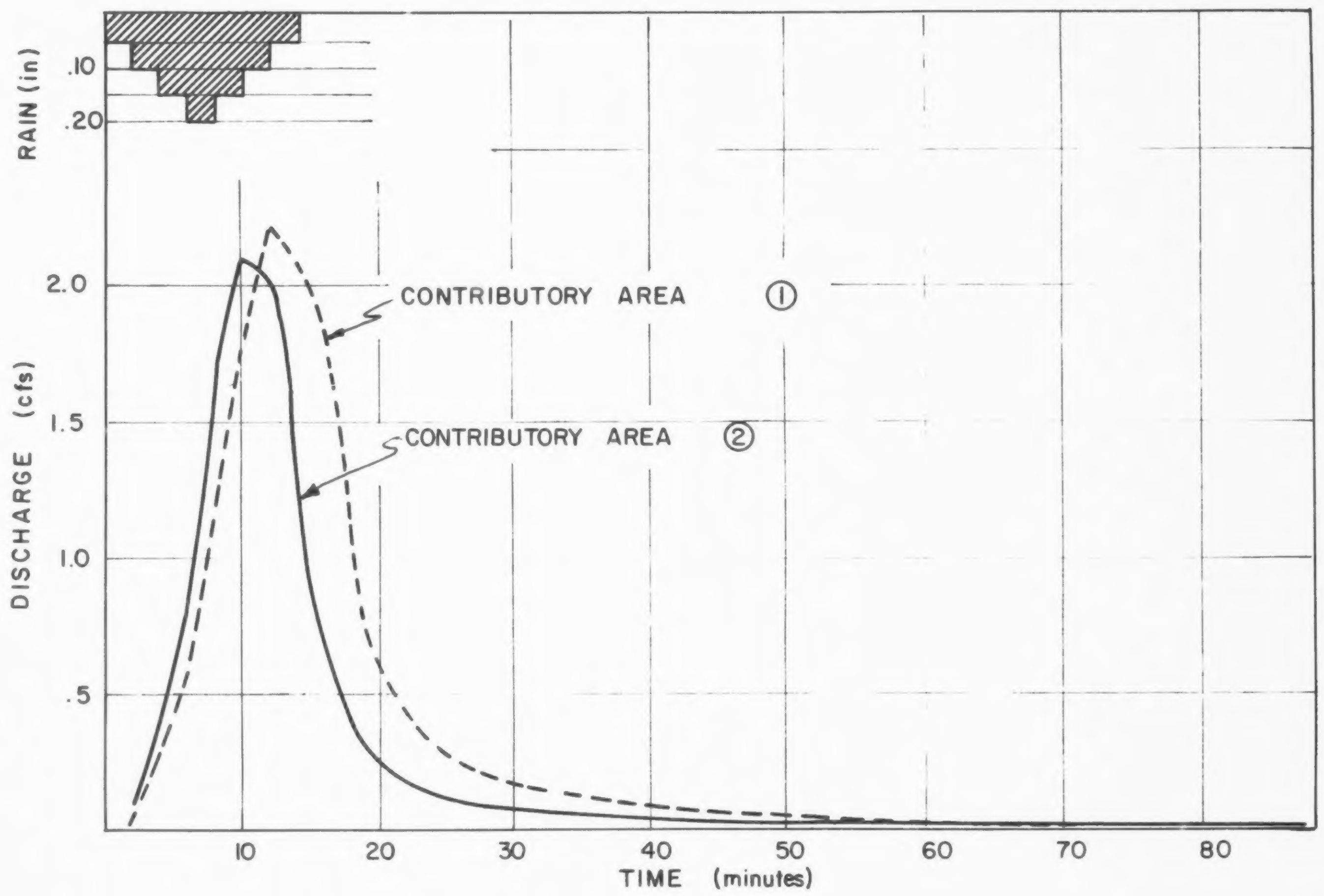


Fig. 21 Design storm rainfall and computed hydrographs

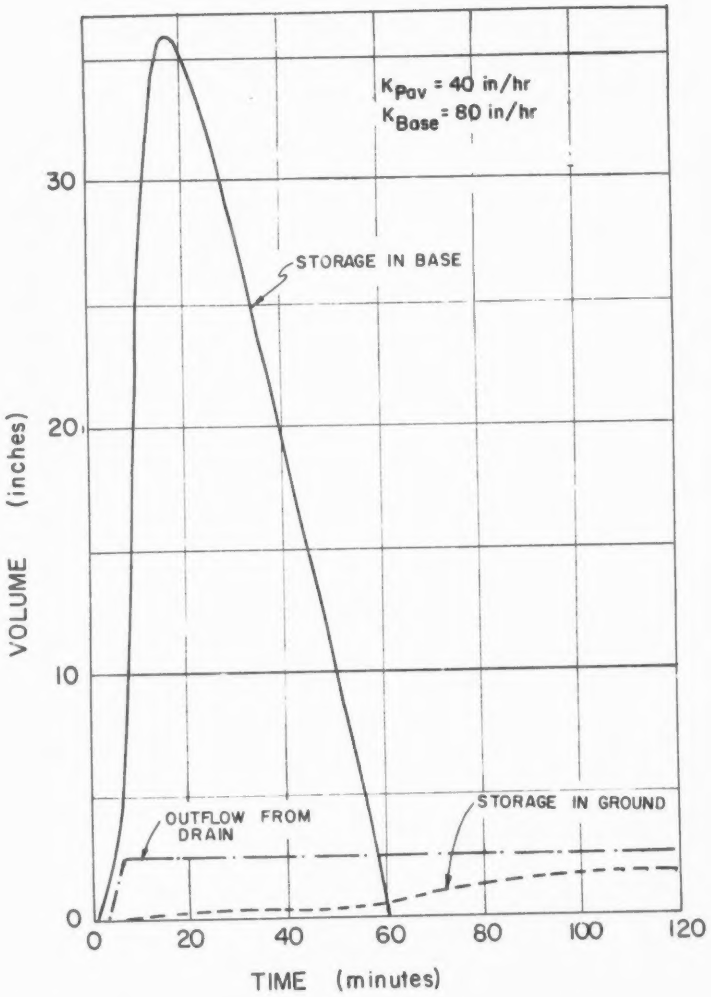


Fig. 22 Storage volumes in porous pavement - high permeabilities

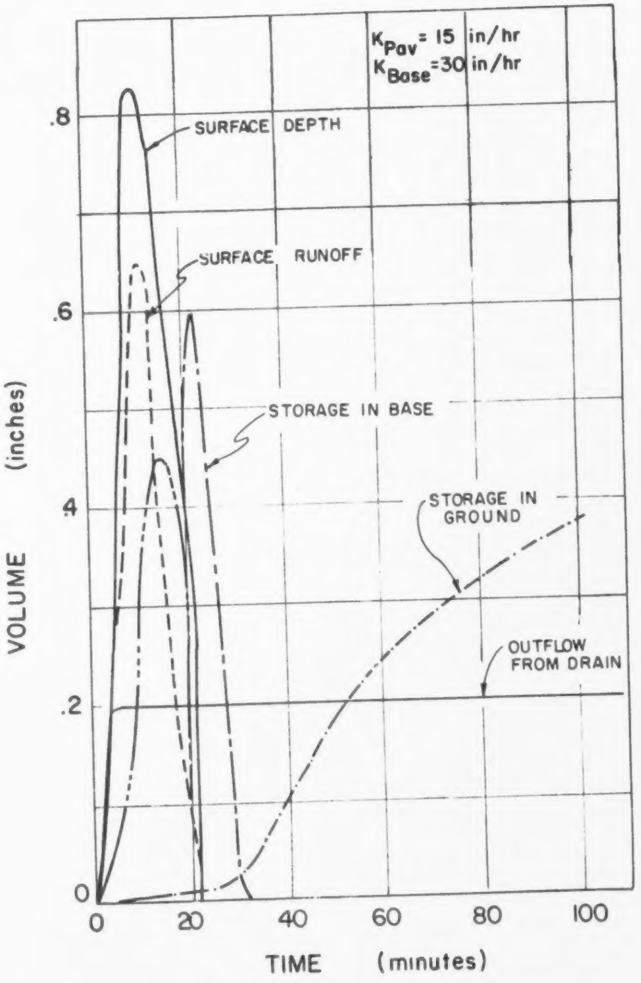


Fig. 23 Storage volumes in porous pavement - low permeabilities

TABLE 4. POROUS PAVEMENT MODELING RESULTS

POROUS PAVEMENT AT THE WOODLANDS
STORM OF 4/08/75 - REDUCED PERMEABILITIES
IN UNITS OF INCHES EXCEPT TIME IN MINUTES

TIME	INFLOW	EVAPOR-ATION DEPTH*	INFLOW TO PAVE	SURFACE OUTFLOW	VERTICAL OUTFLOW FROM PAV	HORIZONTAL OUTFLOW FROM PAV	VERTICAL OUTFLOW FROM BASE	HORIZONTAL OUTFLOW FROM BASE	VERTICAL OUTFLOW THROUGH GROUND	SURFACE STORAGE	STORAGE IN PAV	STORAGE IN BASE	STORAGE IN GROUND
10.	.16	0.00	.16	0.00	.16	0.00	.16	0.00	.08	0.00	0.00	0.00	.08
20.	.27	0.00	.27	0.00	.27	0.00	.27	0.00	.08	0.00	0.00	0.00	.17
30.	.35	0.00	.35	0.00	.35	0.00	.35	0.00	.08	0.00	0.00	0.00	.25
40.	.42	0.00	.42	0.00	.42	0.00	.42	0.00	.08	0.00	0.00	0.00	.33
50.	.47	0.00	.47	0.00	.47	0.00	.47	0.00	.08	0.00	0.00	0.00	.42
60.	.51	0.00	.51	0.00	.51	0.00	.51	0.00	.08	0.00	0.00	0.00	.50
70.	.66	0.00	.66	0.00	.66	0.00	.66	0.00	.08	0.00	0.00	0.00	.58
80.	.75	0.00	.75	0.00	.75	0.00	.75	0.00	.08	0.00	0.00	0.00	.67
90.	.81	0.00	.81	0.00	.81	0.00	.81	0.00	.08	0.00	0.00	0.00	.75
100.	.84	0.00	.84	0.00	.84	0.00	.84	0.00	.08	0.00	0.00	0.00	.83
110.	.87	0.00	.87	0.00	.87	0.00	.87	0.00	.08	0.00	0.00	0.00	.92
120.	.89	0.00	.89	0.00	.89	0.00	.89	0.00	.08	0.00	0.00	0.00	1.00
130.	1.00	0.00	1.00	0.00	1.00	0.00	1.00	0.00	.08	0.00	0.00	0.00	1.08
140.	1.05	0.00	1.05	0.00	1.05	0.00	1.05	0.00	.08	0.00	0.00	0.00	1.17
150.	.75	0.00	.75	0.00	.75	0.00	.75	0.00	.08	0.00	0.00	0.00	1.25
160.	.59	0.00	.59	0.00	.59	0.00	.59	0.00	.08	0.00	0.00	0.00	1.33
170.	1.23	0.00	1.23	0.00	1.23	0.00	1.23	0.00	.08	0.00	0.00	.15	1.42
180.	1.44	0.00	1.44	0.00	1.44	0.00	1.58	0.00	.08	0.00	0.00	.50	1.50
190.	2.43	0.00	2.43	0.00	2.43	0.00	2.93	0.00	.08	0.00	0.00	1.85	1.58
200.	2.59	0.00	2.59	.01	2.50	0.00	4.35	0.00	.08	.08	0.00	3.26	1.67
210.	2.64	0.00	2.72	.03	2.50	0.00	5.76	0.00	.08	.20	0.00	4.68	1.75
220.	2.65	0.00	2.84	.05	2.50	0.00	7.18	0.00	.08	.29	.13	6.00	1.80
230.	1.15	0.00	1.44	.23	1.57	0.00	6.00	0.00	0.00	.59	.75	4.92	1.80
240.	.79	0.00	1.38	.55	.75	0.00	5.67	0.00	0.00	.83	0.00	4.58	1.80
250.	.61	0.00	1.44	0.00	1.44	0.00	6.02	0.00	0.00	0.00	0.00	4.94	1.80
260.	.50	0.00	.50	0.00	.50	0.00	5.43	0.00	0.00	0.00	0.00	4.35	1.80
270.	.42	0.00	.42	0.00	.42	0.00	4.77	0.00	0.00	0.00	0.00	3.69	1.80
280.	.37	0.00	.37	0.00	.37	0.00	4.06	0.00	0.00	0.00	0.00	2.98	1.80
290.	.33	0.00	.33	0.00	.33	0.00	3.30	0.00	0.00	0.00	0.00	2.22	1.80
300.	.29	0.00	.29	0.00	.29	0.00	2.51	0.00	0.00	0.00	0.00	1.43	1.80
310	.27	0.00	.27	0.00	.27	0.00	1.69	0.00	0.00	0.00	0.00	.61	1.80
320.	.24	0.00	.24	0.00	.24	0.00	.85	0.00	0.00	0.00	0.00	0.00	1.80
330.	.22	.00	.22	0.00	.22	0.00	.22	0.00	0.00	0.00	0.00	0.00	1.80
340.	.21	.00	.21	0.00	.21	0.00	.21	0.00	0.00	0.00	0.00	0.00	1.80
350.	.19	.00	.19	0.00	.19	0.00	.19	0.00	0.00	0.00	0.00	0.00	1.80
360.	.18	.00	.18	0.00	.18	0.00	.18	0.00	0.00	0.00	0.00	0.00	1.80
370.	.17	.00	.17	0.00	.17	0.00	.17	0.00	0.00	0.00	0.00	0.00	1.80

* for evaporation rate of 5 inches/30 days evaporation rate is 0.0012 inches/10 minutes

NOTE: (inches) x 2.54 = (centimeters)

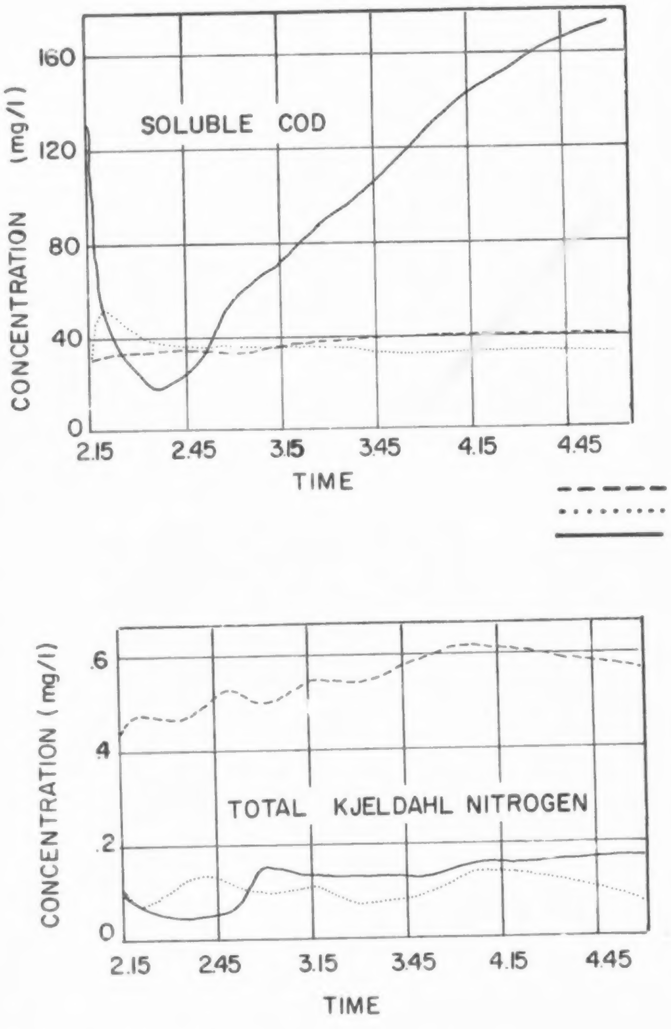
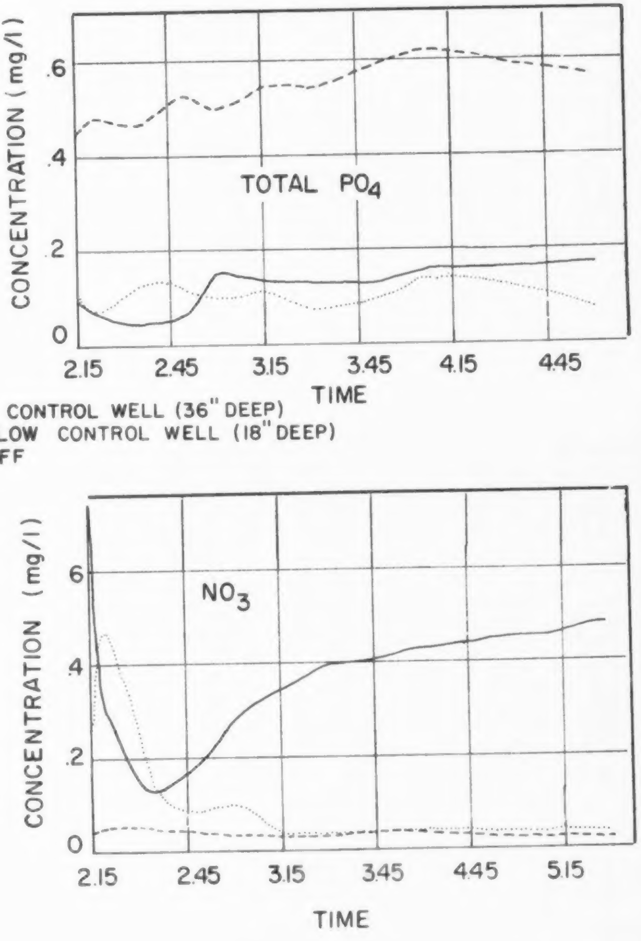


Fig. 24 Water quality in porous pavements - COD and Kjeldahl Nitrogen



- - - - - DEEP CONTROL WELL (36" DEEP)
········· SHALLOW CONTROL WELL (18" DEEP)
——— RUNOFF

Fig. 25 Water quality in porous pavements - Nitrates and Phosphates

TABLE 5 OUTFLOW HYDROGRAPHS FROM POROUS PAVEMENT MODEL

```
* * * POROUS PAVEMENT AT THE WOODLANDS * * * DESIGN STORM* * *


                        SURFACE RUNOFF HYDROGRAPH, IN INCHES

0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000   ,005762
 ,026398   ,054668   ,233323   ,551978  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000
0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000  0,000000


                        HYDROGRAPH OF OUTFLOW THRU DRAIN , IN INCHES

 ,078412   ,188335   ,270478   ,333985   ,384221   ,424582   ,580804   ,666757   ,722477   ,760934
 ,788313   ,807990   ,920954   ,968973   ,665427   ,507186  1,000000  1,000000  1,000000  1,000000
1,000000  1,000000  1,000000  1,000000  1,000000  1,000000  1,000000  1,000000  1,000000  1,000000
1,000000  1,000000   ,741812   ,205656   ,191049   ,178223   ,166866   ,156735   ,147640   ,139429
 ,131977   ,125183   ,975594  1,000000   ,702158   ,519269   ,502389   ,491481   ,480410   ,467751
 ,453279   ,437100   ,419401   ,400375   ,380193   ,359005   ,336936   ,314090   ,290558   ,266414
 ,193939   ,168404   ,150409   ,136672   ,125661   ,116541   ,108804   ,102120   ,096261   ,091067
 ,086415   ,082216   ,078398   ,074907   ,071695   ,068729   ,065976   ,063412   ,061258   ,059253
 ,057383   ,055635   ,053996   ,052458   ,051011   ,049648   ,048362   ,047146   ,045996   ,044907
 ,043874   ,042893   ,041960   ,041074   ,040229   ,039425   ,038658   ,037927   ,037228   ,036561
 ,035923   ,035313   ,034730   ,034171   ,033637   ,033124   ,032634   ,032163   ,031712   ,031279
 ,030864   ,030466   ,030084   ,029717   ,029227   ,028751   ,028289   ,027840   ,027403   ,026977
 ,026564   ,026161   ,025769   ,025386   ,025014   ,024651   ,024297   ,023952   ,023615   ,023287
 ,022966   ,022653   ,022347   ,022046   ,021756   ,021471   ,021192   ,020919   ,020653   ,020392
 ,020136   ,019886   ,019642   ,019402   ,019168   ,018938   ,018713   ,018492   ,018276   ,018064
```

SECTION 7

WATER QUALITY

DATA ANALYSIS

With regard to specific data concerning pollutant availability that may be used to define the required water quality constituents to be used in the SWMM, two studies (1, 18) have been published that provide useful information. These two studies contain street sweeping data and determined the amount of pollutants available per gram of dust and dirt swept from the street. The results are tabulated in Table 6 for various water quality constituents by land-use category.

Obviously there are significant differences between the results of the two studies. Sartor and Boyd report higher values in nearly every instance. An explanation might lie in the differences in experimental methods and geographical areas. Specifically, APWA used only dust and dirt smaller than .32 cm (1/8 in) in Chicago. Furthermore, in the APWA study, the mixed samples were filtered before testing; Sartor and Boyd say only that their samples from 10 different cities were "homogenized." If the latter samples were not filtered, the higher values would have resulted. The coefficients for pollutant availability prediction equations as used in the original SWMM were based primarily on the APWA data. It was concluded that because of the significant variances in observed data, a different approach to water quality prediction was necessary.

Toward this end, an extensive study of water quality data was undertaken. A survey of available literature was made to determine sources of water quality data for storm runoff. Specific information desired was water quality and flow data taken at discrete time intervals during storm events for urban watersheds with seperate sewer systems. Storm hydrographs and pollutographs were found for two watersheds, Third Fork Creek at Durham, North Carolina (16) and K.N. Clapp Drainage Basin at Lubbock, Texas (17). Point sample data collected by the U.S. Geological Survey for the following watersheds were also analyzed:

1) Little Vince Bayou at Pasadena, Texas
2) Willow Waterhole Bayou at Houston, Texas
3) Vince Bayou at Pasadena, Texas

TABLE 6

SUMMARY OF POLLUTANT POTENTIAL OF DUST AND DIRT BY LAND USE

Residential

	BOD mg/g	COD mg/g	PO_4 mg/g	NO_3 mg/g
Sartor & Boyd	11.9	27.75	1.13	.064
APWA	4.3	40.	.05	NA
Average	8.1	31.37	.59	--

Commercial

	BOD mg/g	COD mg/g	PO_4 mg/g	NO_3 mg/g
Sartor & Boyd	8.6	26.	1.03	.6
APWA	7.7	39.	.07	NA
Average	8.15	32.5	.55	--

Industrial

	BOD mg/g	COD mg/g	PO_4 mg/g	NO_3 mg/g
Sartor & Boyd	10.3	53.	1.41	.072
APWA	3.0	--	--	--
Average	6.65	--	--	--

4) Plum Creek at Houston, Texas
5) Brickhouse Gully at Houston, Texas
6) Waller Creek at Austin, Texas

As described in Table 2, the data collected by Rice University consists of a total of 14 storms divided during the three year project duration as follows:

Project Year	Station	No. of Storms	Storm Dates
1	P-30	2	1/18/74, 4/22/74
	H-20	3	3/20/74, 3/26/74, 4/11/74
2	P-10	3	12/05/74, 3/13/75, 4/08/75
	P-30	4	12/05/74, 3/04/75, 3/13/75, 4/08/75
	D-10	3	3/04/75, 3/13/75, 4/08/75
	D-50	3	3/04/75, 3/13/75, 4/08/75
	H-20	1	5/08/75
3	P-10	2	9/05/75, 10/25/75
	P-30	2	9/05/75, 10/25/75
	D-10	2	9/05/75, 10/25/75
	D-50	2	9/05/75, 10/25/75
	H-20	1	6/30/75
	W.S.*	1	6/30/75

*W.S. is station for Westbury Square drainage area.

Data summaries for all the Clapp Basin storms and for all Rice University data are included in Appendix C.

During the first year of this project, the Lubbock and North Carolina data, the USGS point sample data and the data collected by Rice University for the Panther Branch gaging station, P-30, at The Woodlands for the storm of 1/18/74 were analyzed to determine the effect of flow on the concentration of specific water quality constituents. Plots were made which relate the concentration of a given constituent to the instantaneous unit discharge, which is defined as the flow at the time of the sample divided by the drainage area. Plots were developed for suspended solids, BOD, COD, and nitrates. These plots are shown in Figure 26. Table 7 contains the legend showing the plotting symbol used for each watershed.

The suspended solids data shown in Figure 26 show high suspended solids concentration for the Third Fork Creek and for Panther Branch. For Third Fork Creek the high concentrations appear to be due to the fact that the samples were collected by a submersible pump located at the bottom of the stream. Hence during storm events the samples were taken from the lower portion of the flow in the stream. For Panther Branch, the high suspended solids concentrations apparently reflect the large amount of upstream construction activity. In general the data for all the watersheds indicate an increase in suspended solids load as the flow rate increases.

For the BOD data no trend is readily apparent. However, generally the BOD values are less than 10 mg/l. No BOD data were available for Panther Branch. The COD data again show that the Third Fork Creek watershed had significantly higher COD values than other areas. Again this is probably due, at least in part, to the sampling procedure. For the Clapp Basin in Lubbock a relatively consistent trend is shown indicating an increase in COD concentration with increased flow rate.

The nitrate data indicate a relatively consistent increase in concentration as the flow rate increases. It should be noted that the nitrate data for Panther Branch were not plotted on this figure because the concentrations were consistently less than 0.05 mg/l.

The general trend of increasing concentration with increasing flow rate observed in Figure 26 provides an insight to water quality conditions at any instantaneous flow rate but to apply this method to a total hydrograph would assume an unlimited supply of available pollutant. Such is not always the case as evidenced by later data at Station P-10 which indicated a washout of nitrates after a specific volume of flow had been discharged from the watershed. The above inferences were the first indication that perhaps mass of pollutant transported during a storm event would be a more reliable parameter than concentration. This approach was adopted during the latter phases of this effort and, as described at the end of this section, it proved to be the most reliable method available.

Because no definite conclusion regarding water quality predictions could be easily derived, further research became

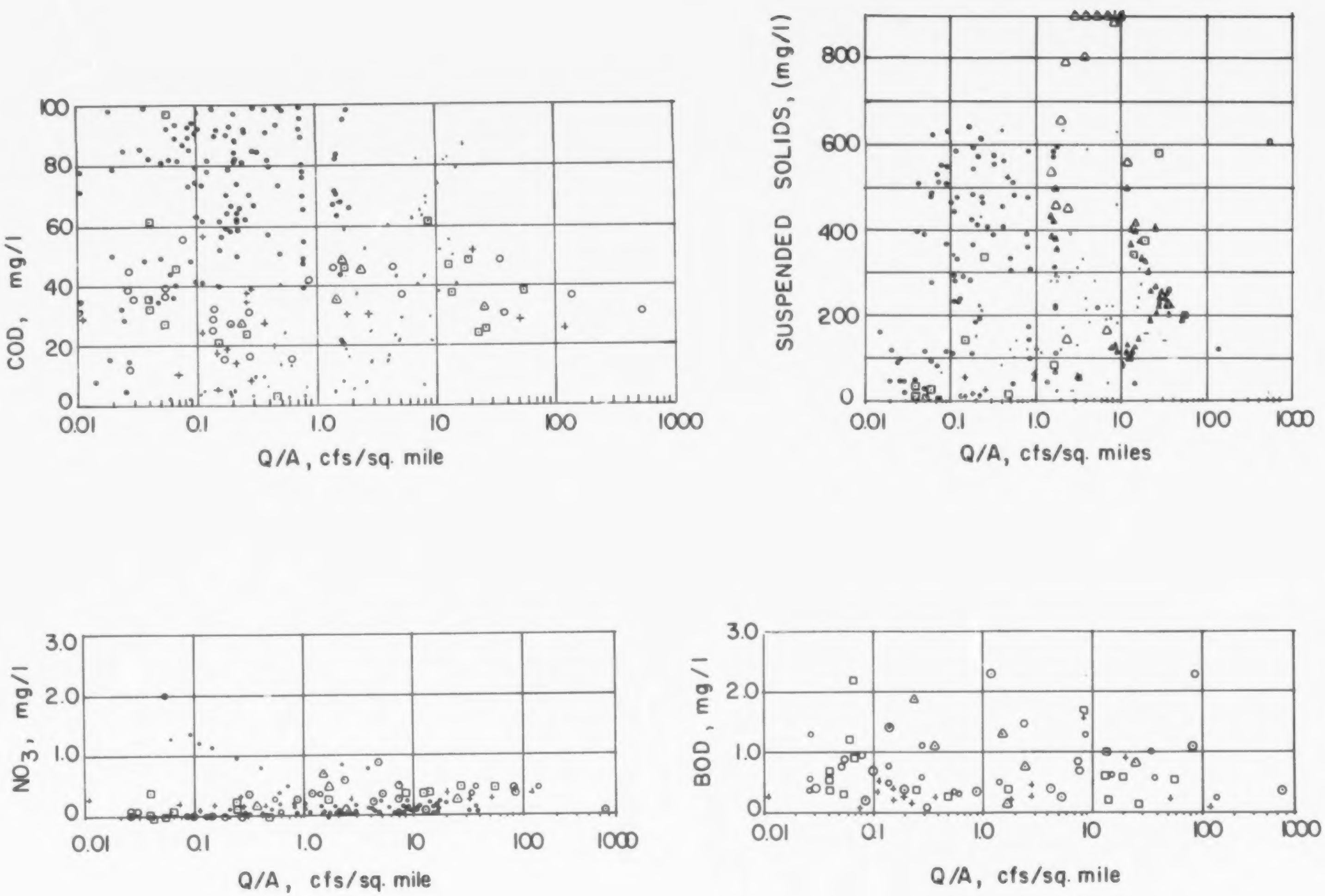


Fig. 26 Unit area discharge relationships

TABLE 7

PLOTTING SYMBOLS FOR UNIT AREA DISCHARGE RELATIONSHIPS
Used in Figure 26

Watershed	Symbol
Little Vince Bayou at Pasadena, TExas	○
Willow Waterhole Bayou at Houston, Texas	⊙
Vince Bayou at Pasadena, Texas	⊡
Plum Creek at Houston, Texas	+
Panther Branch at The Woodlands, Texas	△
Brickhouse Gully at Houston, Texas	◬
Waller Creek at Austin, Texas	⬡
Third Fork Creek at Durham, North Carolina	◦
K. N. Clapp Drainage Basin at Lubbock, Texas	·

necessary. Several different approaches were attempted including the relationships between pollutant loading and the following parameters:

1) Rainfall
    a) Intensity
    b) Duration
    c) Total
2) Runoff
    a) Volume - cu. ft. and in.
    b) Peak flow
    c) Velocity
    d) Rate
3) Land Use
    a) Type
    b) Slope
    c) Curb length
    d) Population density
    e) Impervious cover
    f) Vegetation density

Most of these efforts proved to be fruitless and were discontinued. Only the runoff volume relationship (2a) was judged to be reliable and was subsequently developed in conjunction with pollutant mass loading to the fullest extent possible as described later in this section.

Another approach involved the determination of temporal relationships of pollutants to discharge. Plots of all observed hydrographs and pollutographs on the same time axis were used to compare time at peak and time at specific volume percentages; a sample plot for suspended solids is shown in Figure 27. This analysis did not provide any capability to predict pollutant generation.

Ratios between flow at peak concentration and peak flow, R_Q, and also between time at peak concentration and time at peak flow, R_T, were determined for suspended solids and Kjeldahl nitrogen during selected storms as listed in Table 8. The ratios were not sufficiently constant so no conculsions could be derived.

A correlation between inches of runoff per day and pounds of pollutant per acre was attempted as shown in Table 9 and Figure 28 for nitrates. Although correlation trends were indicated, the results were inconclusive and not applicable to specific storm event water quality predictions.

A final correlation between pounds of pollutant per inch of runoff and peak discharge as shown in Figure 29 for Hunting Bayou indicates the existence of a peak discharge at which the pollutant rate per unit of runoff may be a maximum. But further data is necessary to verify this conclusion. A similar relation-

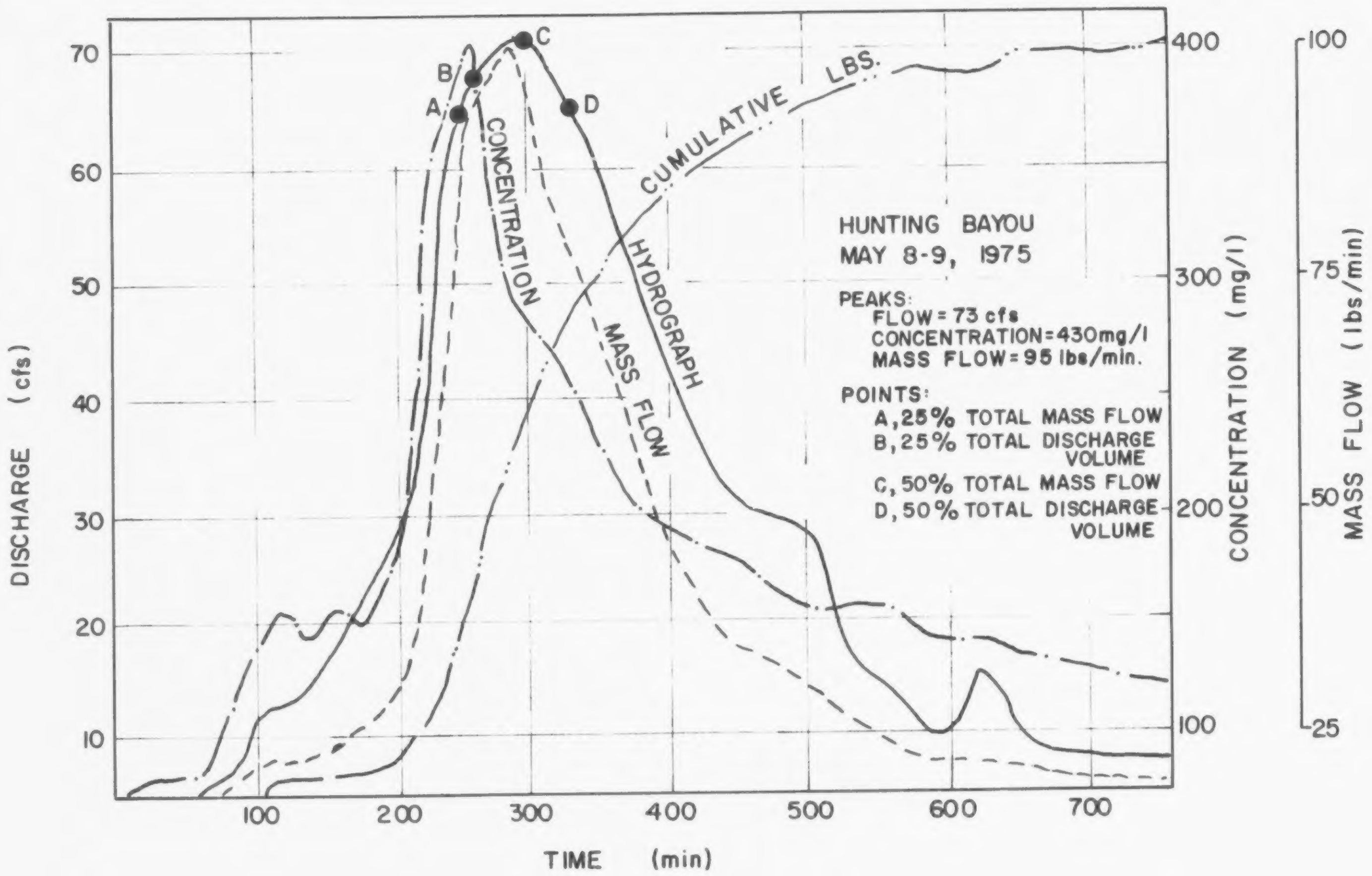


Fig. 27 Temporal relationships of suspended solids to discharge

TABLE 8 DETERMINATION OF R_Q AND R_T RATIOS

SUSPENDED SOLIDS

Date	Station	Peak Flow		Peak Conc.			R_Q	R_T
		Flow	Time	Conc.	Time	Flow		
1/18/74	P-30	1260	1077	2090	293	82	.065	.272
3/26/74	H-20	40	488	376	309	29	.725	.633
4/11/74	H-20	11	466	293	500	11	1.000	1.073
4/22/74	P-30	10	354	1600	477	6	.625	1.347
10/28/74	P-30	380	1831	1256	479	33	.087	.262
12/05/74	P-10	277	1332	130	138	18	.065	.104
12/05/74	P-30	305	1489	720	90	14	.046	.060

KJELDAHL NITROGEN

Date	Station	Peak Flow		Peak Conc.			R_Q	R_T
		Flow	Time	Conc.	Time	Flow		
3/26/74	H-20	40	488	6	64	17	.425	.131
4/11/74	H-20	11	466	3	227	8	.691	.487
4/22/74	P-30	10	354	2	423	7	.730	1.195
12/05/74	P-10	277	1332	1	483	18	.072	.104
12/05/74	P-30	305	1489	2	42	10	.033	.028

TABLE 9 NITRATE YIELD AS A FUNCTION OF DAILY RUNOFF

Date	Station	Yield Rate lb/ac	Daily Runoff in/day
1/18/74	P-30	.009	.37
3/26/74	H-20	.029	.26
4/11/74	H-20	.0058	.096
4/22/74	P-30	.0002	.003
5/08/74	H-20	.020	.3
6/30/74	H-20	.0592	.886
10/28/74	P-30	.0044	.174
12/05/74	P-10	.0017	.183
12/05/74	P-30	.0037	.215
3/13/75	P-10	.0012	.053
4/08/75	P-10	.0018	.538
4/08/75	P-30	.053	.610

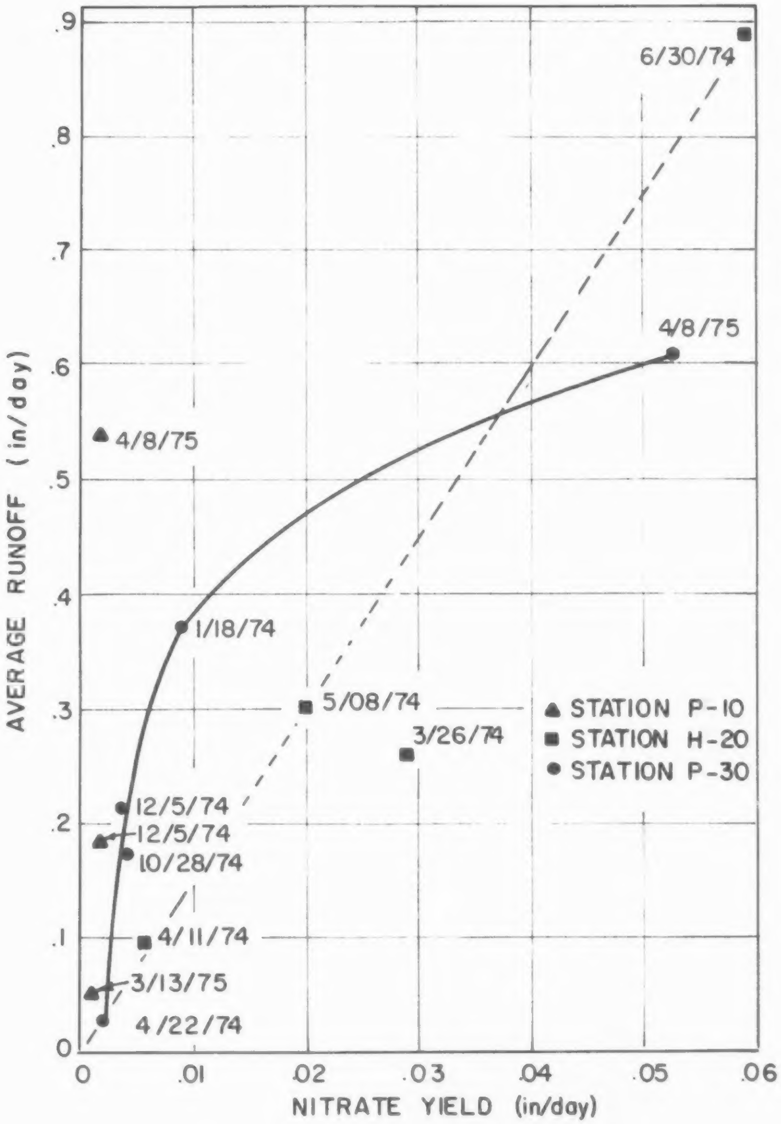


Fig. 28 Nitrate yield as a function of runoff

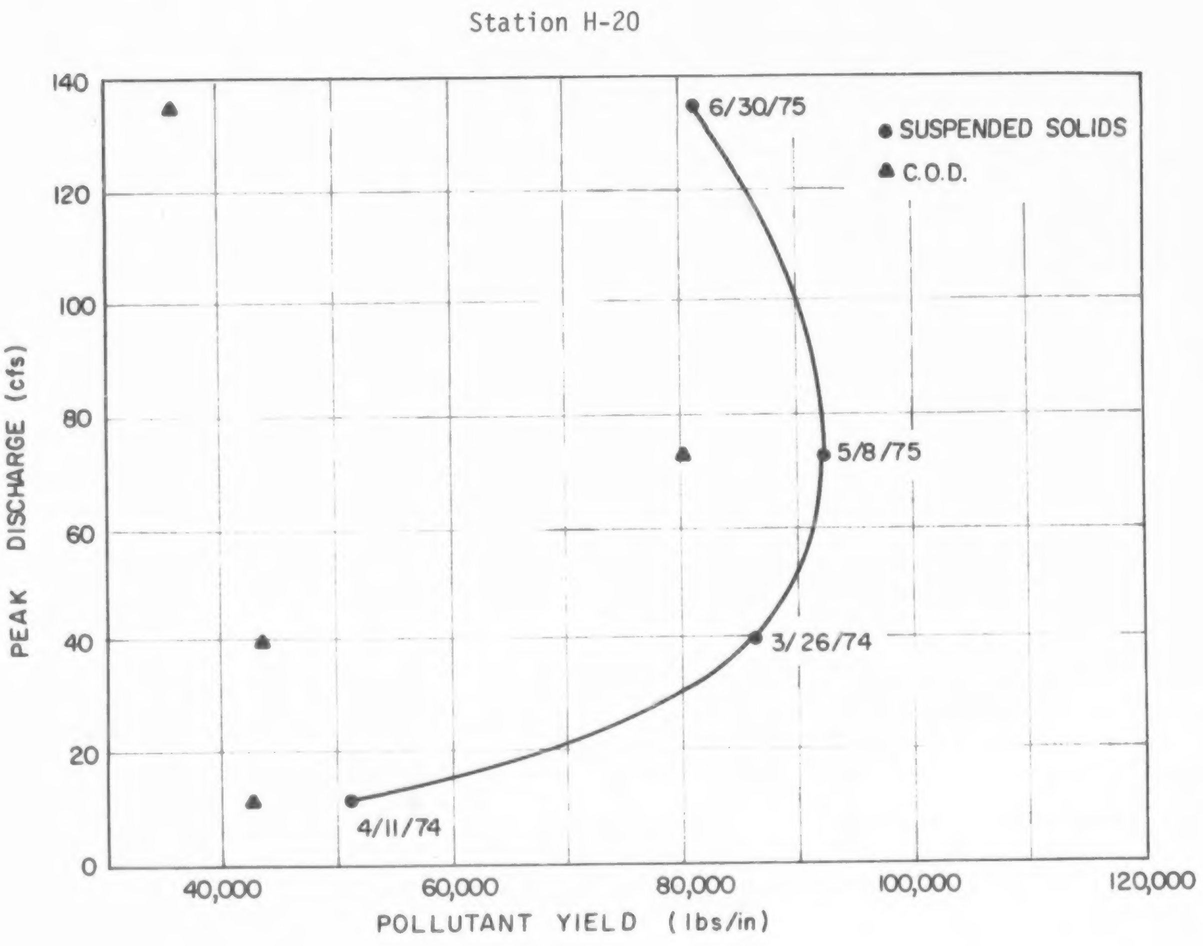


Fig. 29 Pollutant yield as a function of peak discharge

ship between total pounds of pollutant and peak flow proved to be unusable.

As part of this work effort, a computer program for data processing was developed, the source code listing for which is also included in Appendix A. The final output from this program was a line plot of the comparison between two parameters that are user selected. A large number of runs were performed to plot hydrographs, pollutographs, mass flow graphs (loadographs), and comparisons of each pollutant against any other parameter. The last series of runs included plots of the logarithmic transforms of cumulative pounds of pollutant versus cumulative runoff volume in cubic feet for each storm. Because this endeavor seemed to hold promise of success, all the data for Stations P-10, P-30, and H-20 were plotted for these two parameters.

These plots are reproduced in Figures 30 through 34. It was found that a straight line approximation could be fitted to these curves for suspended solids, COD, Kjeldahl nitrogen, nitrates and phosphates. The linear equation coefficients were determined by least squares analysis of data points that were subjectively screened to eliminate extreme values. The corresponding linear equations are also represented in Figures 34 through 34.

A further development was realized when the end point on each cumulative pounds versus cumulative runoff curve was plotted with respect to other similar points with the pollutant load reduced to a unit area. In other words, plots of unit pollutant load in pounds per acre against runoff volume in inches tend to fall on a line whose slope is determined by land use as shown in Figure 35.

COD and Kjeldahl nitrogen exhibit remarkably linear characteristics in Figures 31 and 32. Suspended solids, nitrates and phosphates seem to exhibit some nonlinearity but linearization is still reasonable. As expected, cumulative load will increase with cumulative runoff volume since $M = cQ\ t$

where

M = mass of pollutant
c = concentration of pollutant
Q = discharge
t = time interval

But application of this equation throughout a pollutograph duration would imply a constant concentration, c. This is seldom the case in observed pollutographs.

In Figures 30 through 34 the concentration is made up of two components, a uniform base concentration component, C_o, and

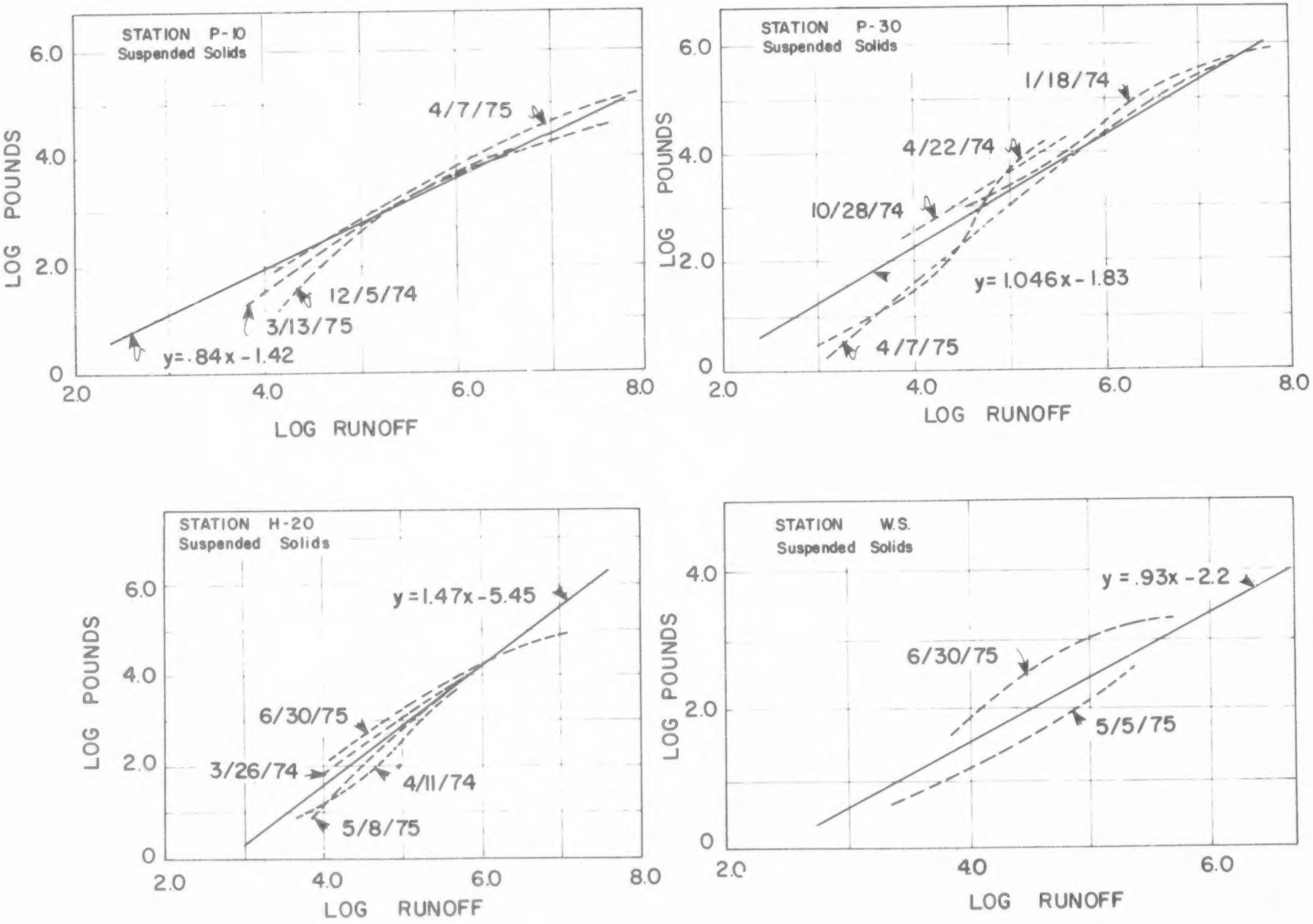


Fig. 30 Water quality relationship - suspended solids

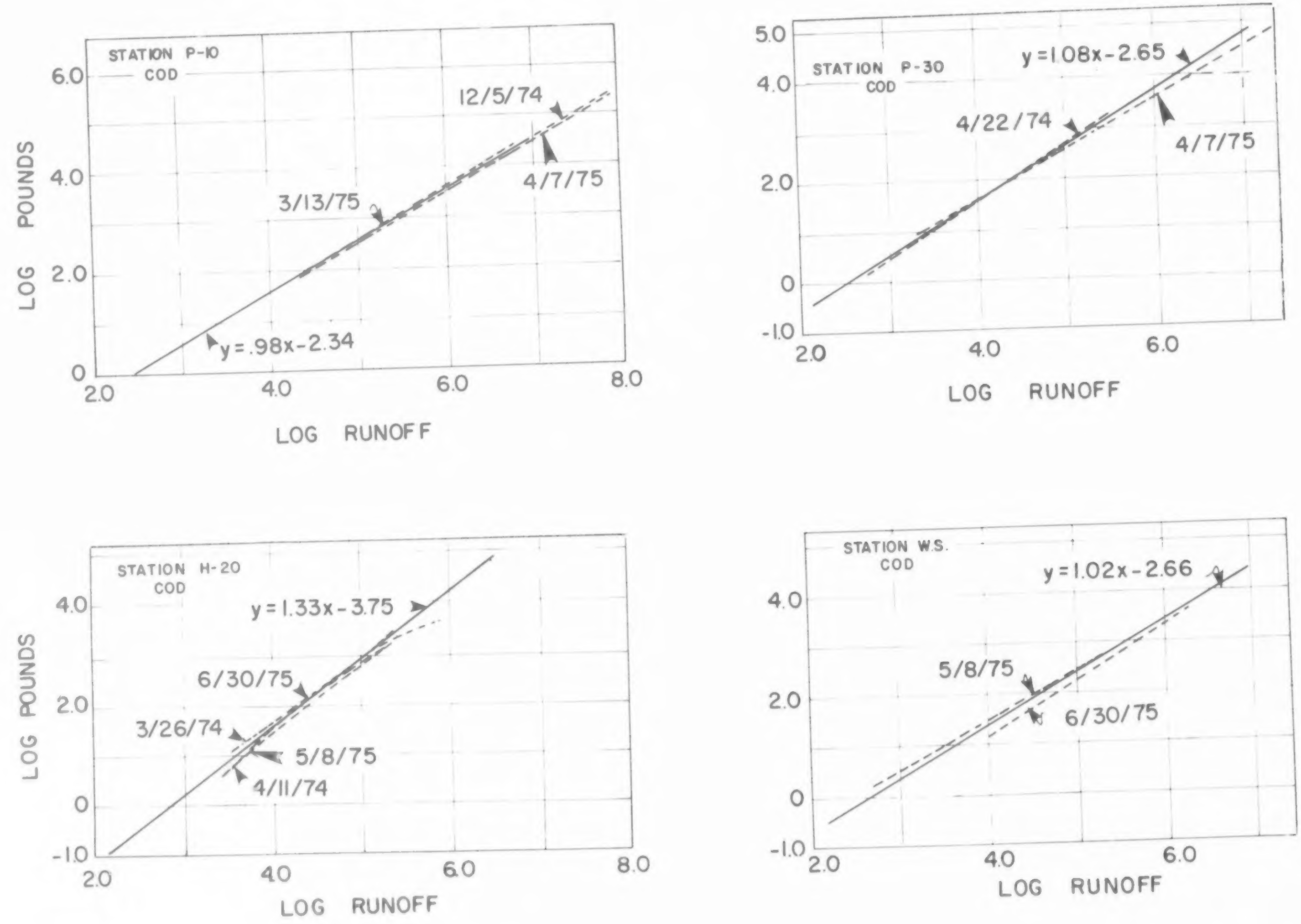


Fig. 31 Water quality relationship - COD

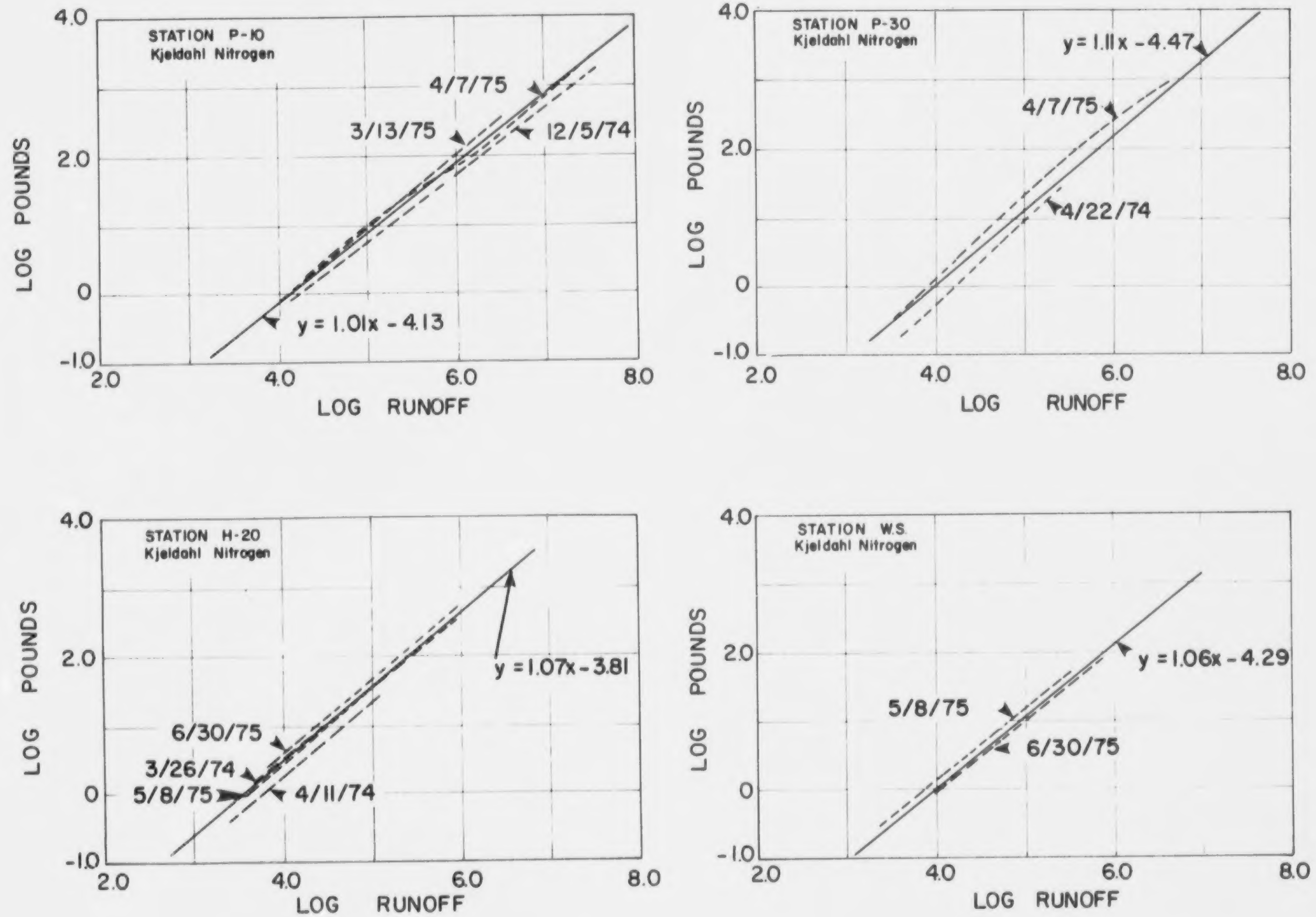


Fig. 32 Water quality relationships - Kjeldahl nitrogen

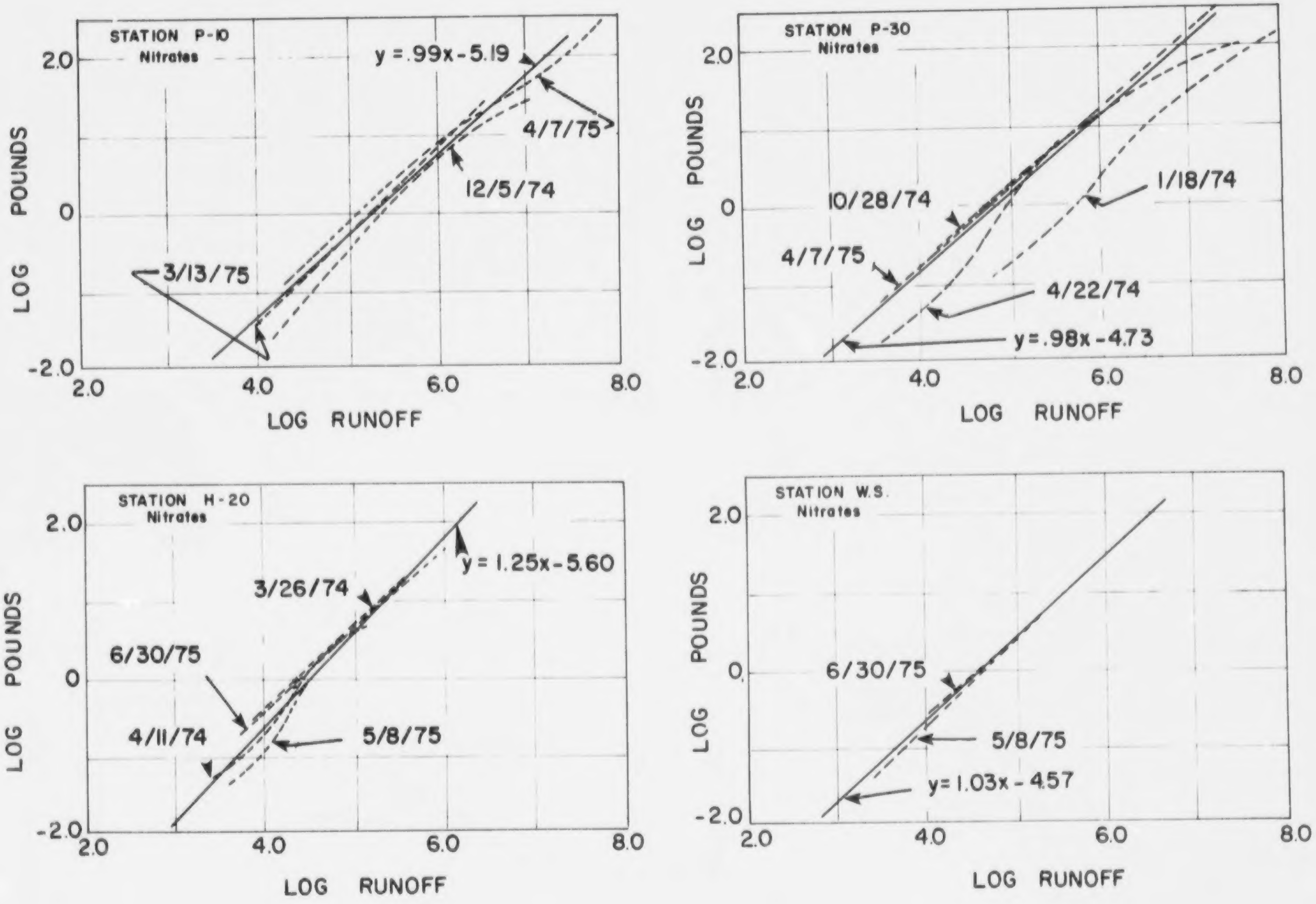


Fig. 33 Water quality relationship - Nitrates

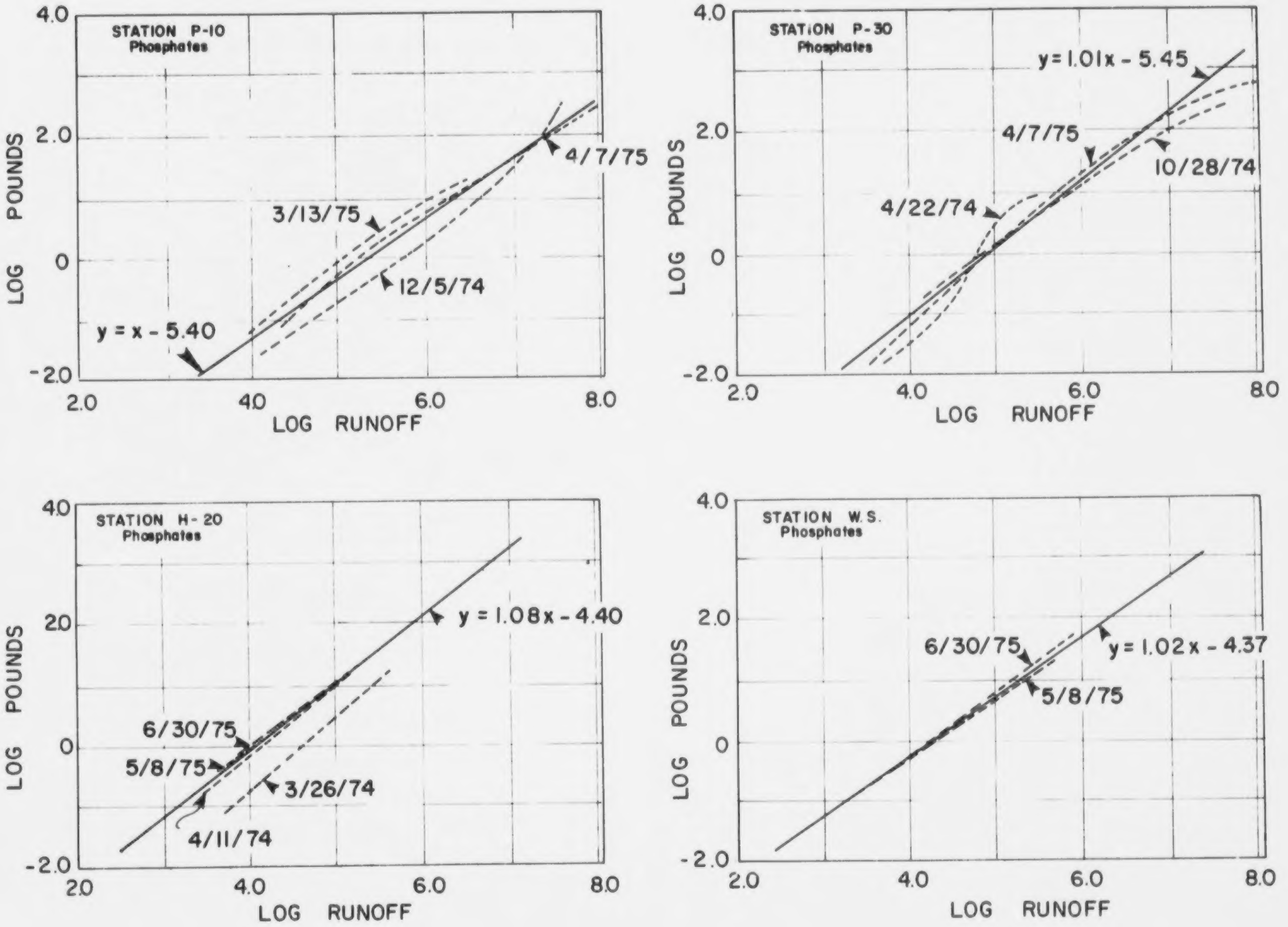


Fig. 34 Water quality relationship - Phosphates

a second concentration exponent, C_1, which varies exponentially with discharge. This relationship can be expressed mathematically as

$$\log M = C_o + C_1 \log (Qt)$$

The uniform base concentration is not an unreasonable concept since most streams do tend to reach a constant concentration during low flow periods.

The paucity of data precludes the derivation of any general conclusions at this time, but it is evident that the modeling of mass flow rates (loadographs) are substantially facilitated by this method.

In spite of the fact that logarithmic transformation is a linearization process, it is believed that pollutant washoff, as well as many other phenomena in nature, is an exponential function of its environmental parameters. Therefore it is not unexpected that the logarithmic transforms would exhibit linear tendencies.

The coefficients of the linear equations, as listed in Table 10, are widely varying at each station and for each parameter; but the exponent (slope) coefficient for P-10 and P-30 is approximately unity. This would indicate a greater dependence upon flow with increasing urbanization.

It should be emphasized that these equations are site specific and further data are necessary before any definite conclusions can be derived.

The relationship between the slopes of the three lines representing pollutant loadings at P-10, P-30 and H-20 is important. P-10 and P-30 have almost similar loading rates with H-20 being greater by several orders of magnitude except in the case of suspended solids and COD where the differences are not as great.

Some inferences regarding urbanization may be derived from Figure 35:

1. The pollutant loading rate in lb/ac per inch of runoff is directly, but not always linearly, proportional to increasing urbanization.

2. The variance in pollutant loading rates is roughly similar for nitrates, phosphates and Kjeldahl nitrogen for each type of land use. In the case of COD the variance is not as marked and for suspended solids it is almost negligible. The latter case may result from severe channel erosion on Panther Branch.

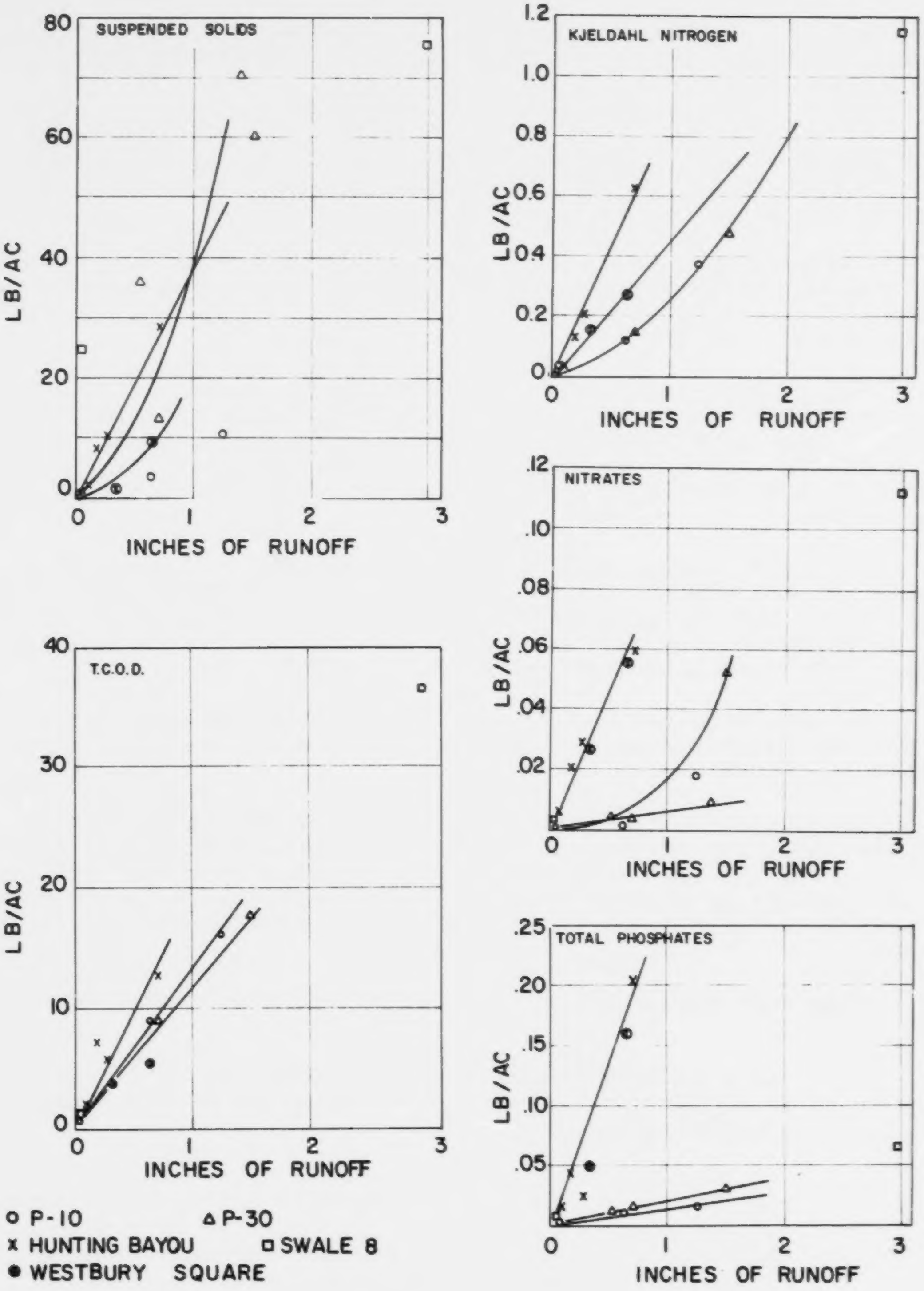


Fig. 35 Total pollutant loadings as a function of runoff

TABLE 10

WATER QUALITY EQUATIONS

Station	Parameter	Equation
P-10	SS	log M = 0.84 log Qt - 1.42
	COD	log M = 0.98 log Qt - 2.34
	KN	log M = 1.01 log Qt - 4.13
	NO_3	log M = 0.99 log Qt - 5.19
	PO_4	log M = 1.00 log Qt - 5.40
P-30	SS	log M = 1.05 log Qt - 1.83
	COD	log M = 1.08 log Qt - 2.65
	KN	log M = 1.11 log Qt - 4.47
	NO_3	log M = 0.98 log Qt - 4.73
	PO_4	log M = 1.01 log Qt - 5.45
H-20	SS	log M = 1.47 log Qt - 5.45
	COD	log M = 1.33 log Qt - 3.75
	KN	log M = 1.07 log Qt - 3.81
	NO_3	log M = 1.25 log Qt - 5.60
	PO_4	log M = 1.08 log Qt - 4.40

3. Both linear and curvilinear relationships are represented indicating a nonlinear dependence of nitrates and Kjeldahl nitrogen to runoff.

4. The one major data point for Swale 8 (D-50) shows that pollutant generation from the Swale 8 watershed is similar to that from the Panther Branch watershed.

The water quality predictive methods discussed in this subsection are a novel approach to this problem. The available data show definite trends but much more data are necessary before general conclusions can be drawn. Consequently the derived relationships are based on insufficient data and should only be used after this restriction has been considered.

The modifications to the SWMM by EH&A to model water quality are based on this new approach. As described in the following subsection, Figures 30 through 35 facilitated the selection of the appropriate coefficients to be used as input to the modified SWMM.

WATER QUALITY MODELING

As stated previously, the present SWMM predicts the concentrations of suspended solids, BOD, total coliform, COD, settleable solids, nitrates, phosphates, and grease in storm runoff. The basic theory used to predict these constituents assumes that the amount of pollutant washed off in any time interval is proportional to the amount remaining on the ground. This results in a first-order differential equation which integrates to the following:

$$P_o - P = P_o\,(1 - e^{-kt})$$

where

P_o is the initial amount of pollutant per unit area

P is the remaining amount of pollutant per unit area at time, t and

k is the decay rate.

In the verification of the water quality predictive capability as described in the initial documentaiton of the SWMM (9), it was found necessary to add an availability factor which yielded the following basic equation which is used with appropriate conversion factors to predict suspended solids and BOD:

$$P_o - P = A_o\,P_o\,(1 - e^{-kt})$$

where A_o is an availability factor which is defined as a percentage of the pollutant amount, P_o, that is available for capture by storm runoff. Coliform densities are predicted directly by multiplying the suspended solids concentration by an appropriate conversion factor. A more detailed discussion of the overall procedure for predicting water quality constituents in the SWMM is provided in the SWMM program documentation (9).

The indeterminate results of water quality data analyses described in the previous subsection indicated that a radical approach to water quality prediction computations in the SWMM was required for this study. Obviously, the dust and dirt accumulation rates developed for Chicago cannot be universally applicable. Also the 4.6 value for the runoff exponent implies identical rainfall intensities and wash off rates for all storms that are modeled. This is not always true, especially in Texas where rainfall intensities have a very wide range. Another complication arose from the fact that in undeveloped areas the lack of streets and curbs makes water quality prediction difficult because dummy curb lengths, based on average feet of curb per acre of drainage area, had to be utilized. Also it was felt that more than 5 land uses were necessary to adequately describe a watershed.

A simplified approach to water quality prediction in SWMM has been developed. Pollutant build up is not considered in the modified SWMM. The revised model does not require input data on dry days, street cleaning frequency, land uses, or curb length. Instead the pollutant availability at the beginning of the storm is input. The user can determine, external to the model, the effect of dry days, street cleaning frequency and land use, while curb length is no longer a parameter. Also the transposition of data to different geographical areas becomes a user option.

Concurrent with the changes described above, the number of land use options has been increased to 20. Because loading rates for each land use are user input, any combination of land uses is feasible. In a developed area, for example, all 20 land uses may be of urban nature. The selection of land uses to any level of detail is therefore possible. However, the program structure presently only allows for transfer from the Runoff to the Transport Block of information on only eight land uses. The pollutant removal factor or wash off exponent, k, is now an input parameter.

The water quality modeling scheme is accessed by using a value of 2 for ISS in Card Group 9 of the Runoff Block. The pollutant removal factor, k, is also input for each pollutant. This allows for model flexibility in the case where pollutants wash off a subcatchment at different rates for the same rainfall or runoff. The loading factors in lbs/ac are input for each land use specified and all pollutants being modeled. As in the original SWMM only one land use per subcatchment is permitted; also, the area of the subcatchment is substituted for curb length on the land use data cards.

The results from the water quality analyses as described earlier have proved to be very helpful in selecting the pollutant loading factors as well as the removal coefficients. The cumulative pounds of pollutant versus cumulative discharge curves, Figures 30 through 34, and the pounds of pollutant per acre of contributing area versus inches of runoff, Figure 35, can be used to select the input data for water quality prediction in the new EH&A version of the SWMM. An estimated runoff rate and the information in Table 11 can be used to determine the removal coefficients.

The loading rates determined from Figures 30 through 34 are only approximate because of minor variability in the data caused by factors not considered in these analyses, e.g., soil characteristics, flora and fauna, etc. The approximate loading rate determined from the figures can be verified by comparison of the modified SWMM output to observed data; this procedure was followed in all applications of the new water quality modeling version.

Because the Transport Block of the SWMM can only route two pollutants and coliform counts, the model would have to be run at least three times at each station to develop output for suspended solids, COD, nitrates, phosphates and Kjeldahl nitrogen. In order to reduce this volume of computer operations, it was decided that Kjeldahl nitrogen would not be routed through the Transport Block. The modeling of Kjeldahl nitrogen, or any other pollutant if desired, is only dependent upon the input data for the EH&A version; for example, if the loading rate and removal coefficient for nitrates are input as data for COD then the COD results from the model may be interpreted as results for nitrates. This new capability of the SWMM enhances its scope of application since any two pollutants (which are or may be treated as conservatives) can now be modeled with every run. For the purposes of this study the two pollutant pairs selected were suspended solids and COD as the first pair, and nitrates and phosphates as the second.

As described in the following section water quality was modeled for an observed storm at each of the three study areas. Calibration of the new water quality model is a relatively simple task. Modeling results based on initial loading rate and removal coefficient estimates are used to refine subsequent loading rates and removal coefficients until the observed pollutograph is reproduced. In applying the model it was determined that pollutograph peak and loading rate were directly proportional while pollutograph shape and removal coefficients were inversely proportional. Consequently, increasing the loading rate by a factor of 2 resulted in a pollutograph peak increase to double the initial peak, and increasing the removal coefficient by a factor of 2 decreased the pollutograph duration to one half the previous duration. The pollutograph peak was

TABLE 11. PERCENT OF CONTAMINANTS REMOVED FROM STREET SURFACES BY RUNOFF RATE AND DURATION

Runoff Rate (in./hr)	Runoff Duration (hr)							
	0.25	0.5	1.0	2.0	3.0	4.0	5.0	6.0
0.1	10.9	20.5	36.9	60.1	74.8	84.1	90.0	>90.0
0.2	20.5	36.9	60.1	84.1	>90.0	>90.0	>90.0	
0.3	29.1	49.8	74.8	>90.0				
0.4	36.9	60.1	84.1					
0.5	43.7	68.3	90.0					
0.6	49.8	74.8	>90.0					
0.7	55.3	80.0						
0.8	60.1	84.1						
0.9	64.5	87.4						
1.0	68.3	90.0						

from: URS Research Company (1974)

raised in conjunction with an increase in the removal coefficient but a general relationship to quantify the rise could not be determined. Therefore the calibration runs are basically a trial and error procedure to determine the ideal loading rate and removal coefficient combination that would reproduce the observed pollutograph. It must be emphasized that the loading rate and removal coefficient thus derived are valid only for the storm used for calibration. Application of these results to other storms is possible only if prevailing antecedent conditions and rainfall-runoff intensities are similar for both storms and if the study areas are identical or at least homogeneous.

In analyzing the water quality modeling results, it became evident that although the pollutographs could be modeled to reproduce observed pollutographs the actual and computed mass transport graphs did not correspond. It was determined that this condition resulted from slight variations in the runoff quantity model results. For example a slight difference between observed and computed runoff rates concurrent to a high rate of mass transport results in a large increase or decrease in pollutant concentrations. Consequently pollutographs are highly dependent on the accuracy of the hydrograph model output.

To improve the modeling of total mass loading of a pollutant, the EH&A version was used to model pollutant mass flow rates. Again the loading rate and decay factor were adjusted to reproduce mass flow rates as determined from the observed flows and concentrations.

The above discussion identifies the need for the user to be completely aware of the modeling objectives. If pollutant mass flow rates or total loadings are desired, then water quality modeling is essentially independent of water quantity modeling; but if pollutant concentrations are desired, then both quantity and quality modeling results determine the accuracy of the pollutant concentrations. Detailed analyses of each storm modeled during this study are included in the following section.

In summary, the modified SWMM water quality model is very capable because of its flexibility in determining pollutant loading from various land uses. The user selected input data determine the relative dependability of the output thus eliminating some of the previous "black box" computations which although generally applicable could not be adequately calibrated for Stations P-10, P-30 and H-20. It is expected that as more water quality data become available the relationships in Figures 31 through 55 will be substantiated and verified.

SECTION 8

MODEL APPLICATION

GENERAL CONSIDERATIONS

The specific types of data which are required as input to the SWMM have been described in Table 1. This input information is a quantified description of the watershed to provide a computational basis for the model. The basic model inputs required are the rainfall hyetograph for the storm to be modeled, a physical description of each subcatchment to be modeled including the drainage area, percent of impervious cover, ground slope, Manning's roughness factors, estimated retention storage for both the pervious and impervious surfaces, and the coefficients to define Horton's soil infiltration equation. Also required are input data to define the hydraulics of the storm sewer system for each subcatchment and for the main sewers or open channels. These inputs include gutter length, slope, bottom width, and roughness coefficient. For sewers and open channels the cross-sectional area and side slopes, channel slope, and roughness factor must be defined. For water quality modeling a code defining the specific land use in each subcatchment as well as the street-cleaning frequency, the number of dry days prior to the storm event, the number of catch-basins per unit area and the quality of their contents must also be specified.

An important portion of the input data concerns the coefficients to be used in Horton's infiltration equation. This equation is used to calculate the infiltration rate of rainfall into the soil as a function of time by Horton's (8) relationship as described in Section 6.

The initial and final infiltration rates, f_i and f_o, and the decay rate, k, were deduced from USGS rainfall-runoff records. This was done by calculating effective infiltration rates for numerous storm events in the Houston area as listed in Appendix C and plotting these values versus storm duration in order to determine a graphical representation of Horton's equation as shown in Figure 36. As expected, it was found that the initial infiltration rate was highly dependent on antecedent soil moisture conditions.

In order to adequately describe the hydraulic efficiency of the drainage system, it was necessary to choose Manning's rough-

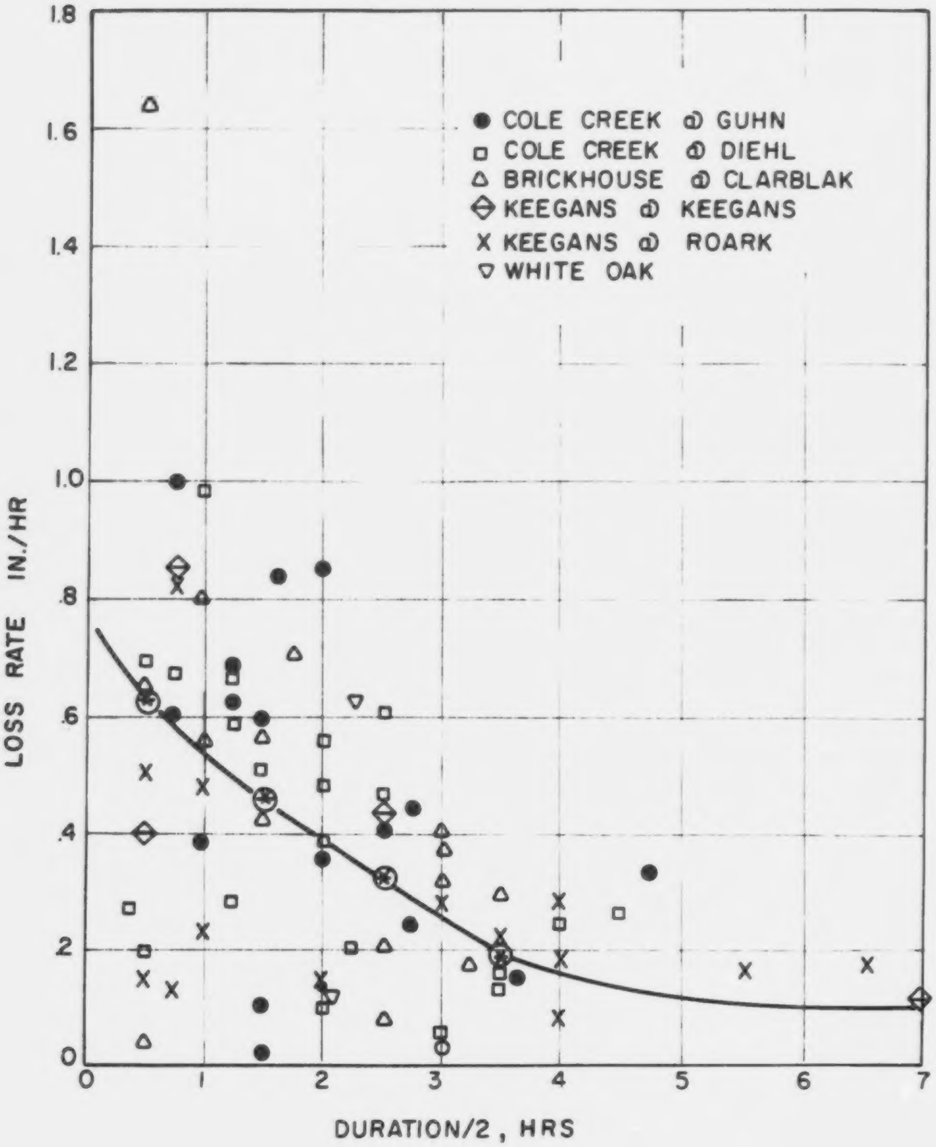


Fig. 36 Infiltration loss rates

ness coefficients for each drainage element. Gutters and open channels were assigned an initial value of 0.10, while a value of 0.03 was used for sewers. These values were adjusted during model calibration to accomodate higher peak flows. Combined sewers are not used in any of the study areas and therefore all initial flow quantity and quality was set equal to zero.

As a prelude to modeling either Hunting Bayou or Panther Branch, specific modeling criteria had to be determined. Several SWMM runs were made with regard to minimizing cost but retaining modeling validity. Data for a dummy watershed consisting of two subcatchments, Table 12, were developed. The SWMM was run for these data at 2, 5, 10, and 20 minute integration time intervals, for average and extreme storm rainfall intensities. The results of this sensitivity analysis, shown in Tables 13 and 14, indicated that there was no substantial gain in modeling accuracy for the 2 and 5 minute integration time interval but the cost increases for modeling at the 2 and 5 minute time intervals was considerable. Consequently, it was determined that a 10 minute time interval was the limit for modeling accuracy at minimum cost, and all further SWMM runs were made with regard to this condition.

HUNTING BAYOU MODELING

Most of the input data concerning the Hunting Bayou drainage system was taken from existing engineering maps of the area, and site inspection of the study area. A map showing the subcatchments and drainage network which were used as input to the model is shown in Figure 37. Five storms were modeled initially. The rainfall data for these storms, listed in Table 15, were obtained from reports published by the U. S. Geological Survey (20,21,22,23). No water quality data was available for these storms because all five storms occurred during 1968 and 1970 prior to the initiation of this project. Consequently, only water quantity was modeled.

As more recent storms on Hunting Bayou were sampled for water quantity and quality, the water quality prediction capabilities of the SWMM were tested. Three storm events have been modeled. Rainfall data for these storms are also listed in Table 15.

The total drainage area of 799.86 hectares (1976.8 acres) is divided into 24 subcatchments ranging in area from 10.11 hectares (25 acres) to 55.85 hectares (138 acres). Other significant subcatchment data are listed in Table 16. Each subcatchment was assigned a land use class for modeling water quality in SWMM. These land use classes are shown in Table 17.

The drainage system includes 23 gutters and pipes in the Runoff Block and 44 manholes and conduits in the Transport

TABLE 12. SWMM INPUT DATA FOR SENSITIVITY ANALYSIS

SUBCATCHMENT DATA

Subcatchment No.	1	2
Width (ft)	4000.0	4000.0
Area (ac)	400.	400.
Percent Imperviousness	.5	1.0
Slope (ft/ft)	0.15	.009
Resistance Factor		
Impervious	.400	.300
Pervious	.250	.250
Surface Storage (in)		
Impervious	.062	.062
Pervious	.184	.184
Infiltration Rate (in/hr)		
Maximum	.75	.75
Minimum	.05	.05
Decay Rate (in/sec)	.00115	.00115
Total Tributary Area (acres)	800.00	

GUTTER AND PIPE DATA

Gutter Number	1	2
Width (ft)	6.0	6.0
Length (ft)	2000.	2000.
Slope (ft/ft)	.001	.001
Side Slopes		
Left	3.0	3.0
Right	3.0	3.0
Manning n	.050	.050
Overflow (in)	90.00	90.00

WATERSHED QUALITY DEFINITIONS

Subarea Number	1	2
Land Use Classification	1	4
Total Gutter Length (100 ft)	20.00	20.00
Number of Catchbasins	0.00	0.00
Number of Constituents	8	
Number of Dry Days	7.0	
Street Cleaning Frequency	0.0	
Passes Per Cleaning	0	
STD Catchbasin Volume (cu ft)	-0.00	
Catchbasin Contents BOD (mg/l)	-0.0	

TABLE 13. SENSITIVITY OF SWMM MODELING FOR CASE A.

Rainfall at 20 minute intervals for Case A, in. .50 1.00 1.50 1.50 1.00 .50
Rainfall at 20 minute intervals for Case B, in. 2.00 4.00 4.00 4.00 3.00 1.00

Total modeling time for all runs is 400 minutes

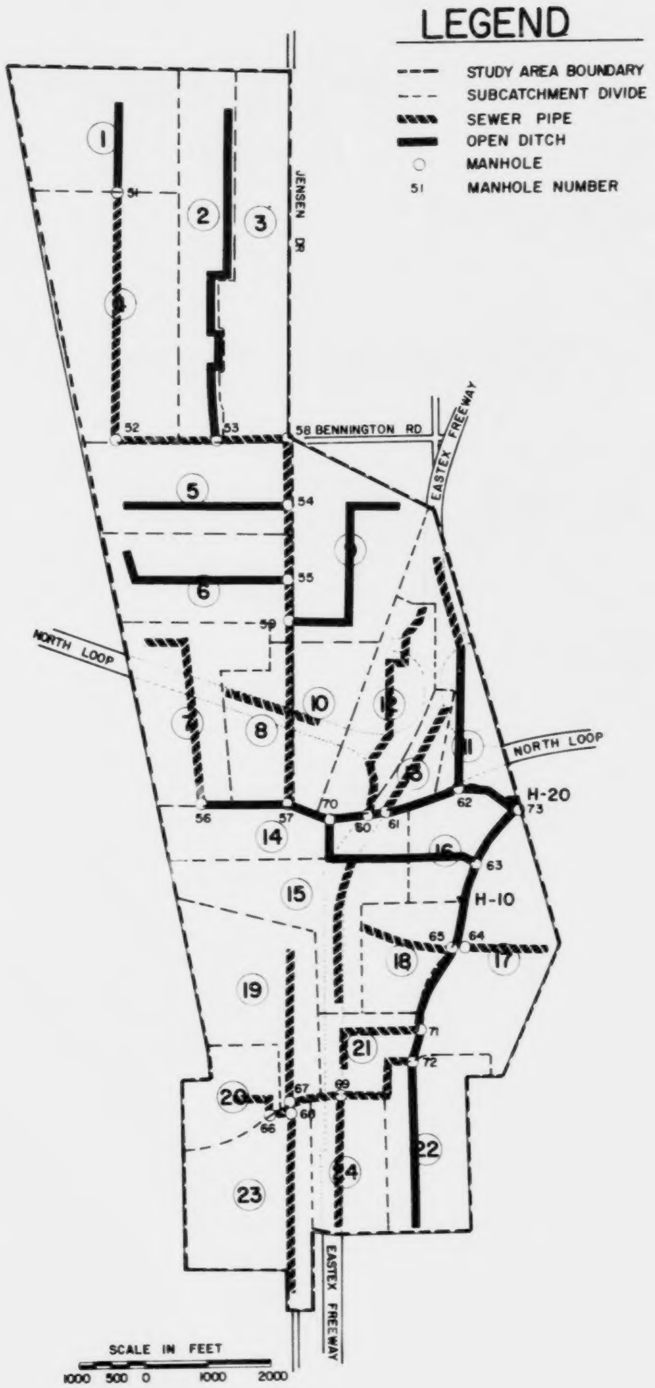
	AT INLET NO. 29				AT INLET NO. 30			
Run Number	A1	A2	A3	A4	A1	A2	A3	A4
Integration Time Interval, min.	2	5	10	20	2	5	10	20
Volumes - Rainfall, 10^5 cu.ft.	58.08	58.08	58.08	58.08				
Runoff, 10^5 cu.ft.	14.45	14.43	14.43	14.48				
Infiltration, 10^5 cu.ft.	17.54	17.55	17.55	17.51				
Peak Value - Flow, cfs	74.50	74.54	74.41	73.50	61.06	61.09	60.94	60.46
BOD, mg/1	.07	.07	.07	.06	.40	.40	.38	.35
Sus. Solids, mg/1	1.18	1.15	1.10	.99	6.78	6.63	6.36	5.82
COD, mg/1	.20	.20	.20	.18	1.29	1.34	1.27	1.17
Time at Peak - Flow, cfs	6.18	6.20	6.20	6.40	6.20	6.20	6.20	6.40
BOD, mg/1	6.02	6.05	6.10	6.20	6.04	6.05	6.10	6.40
Sus. Solids, mg/1	6.06	6.10	6.20	6.40	6.10	6.15	6.20	6.40
COD, mg/1	4.54	4.55	5.00	5.20	4.52	4.50	5.00	5.20
Value at End - Flow, cfs	24.69	24.87	25.15	25.71	23.72	23.87	24.11	24.56
of simulation BOD, mg/1	.02	.02	.02	.02	.13	.14	.14	.14
Sus. Solids, mg/1	.33	.34	.34	.36	2.49	2.51	2.54	2.60
COD, mg/1	.01	.01	.01	.01	.10	.10	.11	.11

TABLE 14. SENSITIVITY OF SWMM MODELING FOR CASE B.

Rainfall at 20 minute intervals for Case A, in.	.50	1.00	1.50	1.50	1.00	.50
Rainfall at 20 minute intervals for Case B, in.	2.00	4.00	4.00	4.00	3.00	1.00

Total modeling time for all runs is 400 minutes

	AT INLET NO. 29				AT INLET NO. 30			
Run Number	A1	A2	A3	A4	A1	A2	A3	A4
Integration Time Interval, min.	2	5	10	20	2	5	10	20
Volumes - Rainfall, 10^5 cu.ft.	174.24	174.24	174.24	174.24				
Runoff, 10^5 cu.ft.	14.48	14.48	14.48	14.48				
Infiltration, 10^5 cu.ft.	105.41	105.41	105.41	105.40				
Peak Value - Flow, cfs	518.64	518.90	518.48	505.38	437.32	435.76	434.62	429.60
BOD, mg/l	.14	.13	.12	.10	.88	.81	.77	.60
Sus. Solids, mg/l	2.45	2.37	2.15	1.79	15.28	14.95	14.18	11.13
COD, mg/l	.20	.20	.25	.24	1.32	1.30	1.61	1.60
Time at Peak - Flow, cfs	5.58	6.00	6.00	6.00	6.02	6.05	6.10	6.20
BOD, mg/l	5.04	5.10	5.10	5.20	5.10	5.15	5.20	5.20
Sus. Solids, mg/l	5.06	5.10	5.20	5.20	5.12	5.15	5.20	5.40
COD, mg/l	4.26	4.25	4.20	4.40	4.24	4.30	4.20	4.40
Value at End - Flow, cfs	93.49	94.22	95.43	97.99	101.82	102.52	103.71	106.15
of Simulation BOD, mg/l	.00	.00	.00	.00	.00	.00	.00	.00
Sus. Solids, mg/l	.00	.00	.00	.00	.00	.00	.00	.00
COD, mg/l	.00	.00	.00	.00	.00	.00	.00	.00


LEGEND
STUDY AREA BOUNDARY
SUBCATCHMENT DIVIDE
SEWER PIPE
OPEN DITCH
MANHOLE
51 MANHOLE NUMBER
JENSEN DR.
58 BENNINGTON RD
EASTEX FREEWAY
NORTH LOOP
NORTH LOOP
H-20
H-10
EASTEX FREEWAY
1
2
3
4
5
6
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SCALE IN FEET
1000 500 0 1000 2000

Fig. 37 Subcatchments and drainage network - Hunting Bayou

TABLE 15. RAINFALL DATA, HUNTING BAYOU WATERSHED

Date of Storm	9/8/68	9/17/68	11/5/68	10/22/70	11/9/70	3/26/74	5/8/75	6/30/75
Time Interval	10	10	15	30	10	30	30	10
	.56	.43	.10	1.50	.49	.12	.05	.15
	.48	.43	.90	1.15	.48	.13	.05	.00
	.41	.44	.40	.60	.47	.00	.05	.00
	.25	.29	.40	.06	.04	.00	.05	.00
	.15	.20	.28	.20	.03	.00	.05	.20
	.05	.12	.07		.03	.00	.10	.20
	.01	.01				.00	.25	.00
						.00	.05	.00
						.00		.00
						.00		.00
						.00		.10
						.08		.10
						.07		.45
						.00		.10
						.00		.10
						.00		.25
						.00		.08
						.04		.08
						.04		.03
						.04		.02
						.04		
						.00		
						.00		
						.00		
						.05		
						.05		
						.05		
Total Rainfall (in)	2.15	1.91	1.92	1.54	3.51	.71	.65	1.86

Note: Total rainfall will not correspond to total storm rainfall when only a partial storm is modeled.

TABLE 16. SUBCATCHMENT DATA, HUNTING BAYOU WATERSHED

					Resistance Factor		Surface Storage (in)		Infiltration Rate (in/hr)		(in/sec)
Subcatchment No.	Width (ft)	Area (ac)	Percent Imperv.	Slope (ft/ft)	Imperv.	Perv.	Imperv.	Perv.	Max.	Min.	Decay
1	2500.0	114.	18.0	.001	.013	.250	.062	.000	.75	.01	.00115
2	800.0	100.	18.0	.001	.013	.250	.062	.000	.75	.01	.00115
3	900.0	138.	25.0	.002	.013	.250	.062	.000	.75	.01	.00115
4	1150.0	89.	18.0	.002	.013	.250	.062	.000	.75	.01	.00115
5	3200.0	109.	18.0	.001	.013	.250	.062	.000	.75	.01	.00115
6	2800.0	94.	23.0	.001	.013	.250	.062	.000	.75	.01	.00115
7	1500.0	106.	24.0	.002	.013	.250	.062	.000	.75	.01	.00115
8	1000.0	55.	18.0	.002	.013	.250	.062	.000	.75	.01	.00115
9	2000.0	119.	18.0	.002	.013	.250	.062	.000	.75	.01	.00115
10	1000.0	56.	25.0	.002	.013	.250	.062	.000	.75	.01	.00115
11	700.0	72.	13.0	.002	.013	.250	.062	.000	.75	.01	.00115
12	1100.0	77.	29.0	.002	.013	.250	.062	.000	.75	.01	.00115
13	600.0	25.	33.0	.002	.013	.250	.062	.000	.75	.01	.00115
14	750.0	61.	18.0	.001	.013	.250	.062	.000	.75	.01	.00115
15	1000.0	84.	18.0	.001	.013	.250	.062	.000	.75	.01	.00115
16	1700.0	47.	17.0	.001	.013	.250	.062	.000	.75	.01	.00115
17	1700.0	118.	29.0	.002	.013	.250	.062	.000	.75	.01	.00115
18	1700.0	53.	29.0	.002	.013	.250	.062	.000	.75	.01	.00115
19	2000.0	103.	29.0	.001	.013	.250	.062	.000	.75	.01	.00115
20	1600.0	45.	29.0	.003	.013	.250	.062	.000	.75	.01	.00115
21	1000.0	39.	29.0	.002	.013	.250	.062	.000	.75	.01	.00115
22	2700.0	96.	29.0	.001	.013	.250	.062	.000	.75	.01	.00115
23	2000.0	109.	29.0	.001	.013	.250	.062	.000	.75	.01	.00115
24	2700.0	66.	29.0	.001	.013	.250	.062	.000	.75	.01	.00115

Total Tributary Area (acres), 1976.80

TABLE 17

LAND USE DATA, HUNTING BAYOU WATERSHED

Subarea Number	Land Use Class.	Total Gutter Length (100 ft)	Number of Catchbasins
1	1	59.00	0.00
2	1	64.00	0.00
3	1	24.00	0.00
4	1	79.00	0.00
5	3	96.00	0.00
6	1	155.00	0.00
7	1	61.00	0.00
8	3	30.00	0.00
9	1	90.00	0.00
10	3	35.00	0.00
11	5	22.00	0.00
12	3	31.00	0.00
13	3	20.00	0.00
14	1	45.00	0.00
15	5	68.00	0.00
16	1	55.00	0.00
17	1	134.00	0.00
18	1	60.00	0.00
19	3	99.00	0.00
20	3	35.00	0.00
21	1	41.00	0.00
22	1	113.00	0.00
23	3	86.00	0.00
24	1	78.00	0.00

Land Use Class is defined as follows:

1. Single and multi-family residential areas
3. Business and commercial activity areas
5. Undeveloped urban open land

Block. Table 18 lists all the gutter and pipe data and Table 19 shows the Transport element characteristics.

The subcatchment, gutter and pipe, and transport element data were essentially identical for all runs. The storm related data include rainfall, Table 15, and infiltration coefficients, Table 20. Infiltration rates were estimated from the observed rainfall and runoff. Initially, the antecedent precipitation index was used to establish a starting infiltration rate but this parameter proved to be unreliable as a deterministic predictor and was subsequently discarded. Therefore all infiltration rates were originally estimated and then calibrated by consecutive modeling runs.

Comparisons of observed and computed hydrographs for five storm events which were modeled are presented in Figures 38, 39 and 40. A comparison of observed and computed total runoff volumes and peak flow rates is shown in Table 21. The overall agreement is reasonable. The average absolute error in volume of runoff was 26% of the observed value, while the average error in peak flow prediction was 20% of the observed peak. The temporal agreement of the hydrographs was very good. For instance, the times of peak flow agreed within ten minutes in four of the five instances. The average error was twenty-two minutes. However, the computed values tended to predict faster returns to low flow conditions than were actually observed.

The original SWMM water quality predictions for the storm of 5/8/75 could not reproduce the observed data; Figure 40 shows the observed and computed results for suspended solids at Station H-20. The EH&A version, with loading rates as shown in Table 22, was run for the same storm and the predicted results for suspended solids, COD, nitrates and total phosphorus are compared to observed data in Figures 41 through 44 and in Table 23. As discussed in the preceding section, even if the pollutographs are adequately reproduced the pollutant mass transport rates are not sufficiently high. Consequently the pollutant mass transport rates were computed and compared to pollutant mass transport rates as determined from the observed discharge rates and concentrations. The total observed and computed pounds of pollutant transported during the storm are also compared in Table 23.

As seen in Figures 41 through 44, the rather large temporal difference between observed and computed pollutographs is minimized in the pollutant mass transport rate graphs; the reductions ranged from 75% for suspended solids to 100% for nitrates. With the exception of COD, the predictions of total pounds of pollutant removed were also improved by the use of mass flow rate graphs. In the case of COD, an extremely high pollutant removal factor seems to be necessary to reduce the total pounds prediction. No physical explanation for this phenomenon could

TABLE 18. GUTTER AND PIPE DATA, HUNTING BAYOU WATERSHED

Gutter Number	Description	Width (ft)	Length (ft)	Slope (ft/ft)	Side Slopes L	Side Slopes R	Manning n	Overflow (in)
1	Ditch	3.0	1450.	.003	1.0	1.0	.100	99.00
2	Ditch	3.0	5700.	.002	3.0	3.0	.100	99.00
3	Pipe	7.5	3900.	.002	-0.0	-0.0	.029	90.00
4	Ditch	4.0	2500.	.001	3.0	3.0	.100	99.00
5	Ditch	4.0	2400.	.001	3.0	3.0	.100	99.00
6	Pipe	6.0	3200.	.004	-0.0	-0.0	.029	72.00
7	Pipe	3.2	2500.	.006	-0.0	-0.0	.029	38.00
8	Ditch	4.0	2200.	.002	3.0	3.0	.100	99.00
9	Pipe	2.0	2500.	.002	-0.0	-0.0	.029	24.00
10	Ditch	5.0	3700.	.003	1.0	1.0	.100	99.00
11	Pipe	5.0	3500.	.003	-0.0	-0.0	.029	60.00
12	Pipe	3.5	2000.	.002	-0.0	-0.0	.029	42.00
13	Ditch	4.0	1800.	.001	3.0	3.0	.100	99.00
14	Pipe	5.5	4500.	.002	-0.0	-0.0	.029	66.00
15	Ditch	4.0	1000.	.001	3.0	3.0	.100	99.00
16	Pipe	4.0	1400.	.001	-0.0	-0.0	.029	48.00
17	Pipe	3.5	1500.	.002	-0.0	-0.0	.029	42.00
18	Pipe	3.5	2500.	.001	-0.0	-0.0	.029	42.00
19	Pipe	3.0	900.	.004	-0.0	-0.0	.029	36.00
20	Pipe	5.0	2100.	.007	-0.0	-0.0	.029	60.00
21	Ditch	4.0	2700.	.001	6.0	6.0	.100	99.00
22	Pipe	3.5	2800.	.001	-0.0	-0.0	.029	42.00
23	Pipe	4.0	2000.	.001	-0.0	-0.0	.029	48.00

TABLE 19. TRANSPORT ELEMENT CHARACTERISTICS, HUNTING BAYOU WATERSHED

Ext.Ele. Number	Description	Slope (ft/ft)	Distance (ft)	Manning n	Geom1 (ft)	Geom2 (ft)	Geom3 (ft)	# of Barrels	AFull (sq ft)	QFull (cfs)	QMax (cfs)
52	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
1	Rectangular	.0010	1500.0	.0290	6.0	8.0	-0.0	1.0	48.000	111.708	134.498
53	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
2	Rectangular	.0015	1000.0	.0290	6.0	8.0	-0.0	2.0	48.000	136.814	164.726
58	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
3	Rect.- Triang.	.0014	1100.0	.1000	10.0	20.0	3.0	1.0	170.000	201.397	265.148
54	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
4	Rect.- Triang.	.0014	1100.0	.1000	10.0	10.0	3.0	1.0	170.000	201.397	265.148
55	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
5	Rect.- Triang.	.0013	600.0	.1000	10.0	20.0	3.0	1.0	170.000	194.071	255.503
59	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
6	Rectangular	.0010	1400.0	.0290	10.0	9.0	-0.0	1.0	90.000	259.818	299.870
51	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
21	Rectangular	.0053	300.0	.0290	10.0	9.0	-0.0	1.0	90.000	598.147	690.353
74	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
22	Rectangular	.0007	1200.0	.0290	10.0	9.0	-0.0	1.0	90.000	217.380	250.889
57	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
56	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
7	Rectangular	.0011	1400.0	.0290	10.5	8.0	-0.0	2.0	84.000	248.490	281.940
8	Alpha	.0014	700.0	.1000	16.0	-0.0	-0.0	1.0	257.229	301.894	301.894
70	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
9	Alpha	.0010	500.0	.1000	16.0	-0.0	-0.0	1.0	257.229	255.147	255.147
60	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
61	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
10	Alpha	.0015	400.0	.1000	16.0	-0.0	-0.0	1.0	257.229	312.490	312.490
11	Alpha	.0010	1200.0	.1000	16.0	-0.0	-0.0	1.0	257.229	255.147	255.147
62	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
12	Alpha	.0018	1100.0	.1000	16.0	-0.0	-0.0	1.0	257.229	342.315	342.315
73	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
66	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
14	Circular	.0250	400.0	.0290	6.0	-0.0	-0.0	1.0	28.274	300.985	325.064
67	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
15	Circular	.0013	800.0	.0290	6.0	-0.0	-0.0	1.0	28.274	68.635	74.126
69	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
16	Circular	.0033	1600.0	.0290	7.0	-0.0	-0.0	1.0	38.485	164.952	178.148
72	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
17	Beta	.0030	500.0	.1000	7.5	-0.0	-0.0	1.0	146.751	250.156	250.156
71	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
18	Beta	.0014	1400.0	.1000	7.5	-0.0	-0.0	1.0	146.751	170.889	170.889
65	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
64	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
19	Beta	.0007	4400.0	.1000	8.0	-0.0	-0.0	1.0	166.970	143.530	143.530
63	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
20	Beta	.0070	1100.0	.1000	8.0	-0.0	-0.0	1.0	166.970	453.882	453.882

TABLE 20. INFILTRATION PARAMETERS, HUNTING BAYOU WATERSHED

STORM DATE	STATION	INFILTRATION RATES Initial in/hr	Final in/hr	Decay /sec
9/08/68	H-10	1.00	0.10	.0005
	H-20	1.00	0.10	.0005
9/17/68	H-10	0.75	0.10	.0005
	H-20	0.75	0.10	.0005
11/09/70	H-10	2.50	0.10	.0005
	H-20	2.50	0.10	.0005
3/26/74	H-20	0.10	0.02	.0005
5/08/75	H-20	0.30	0.10	.0005

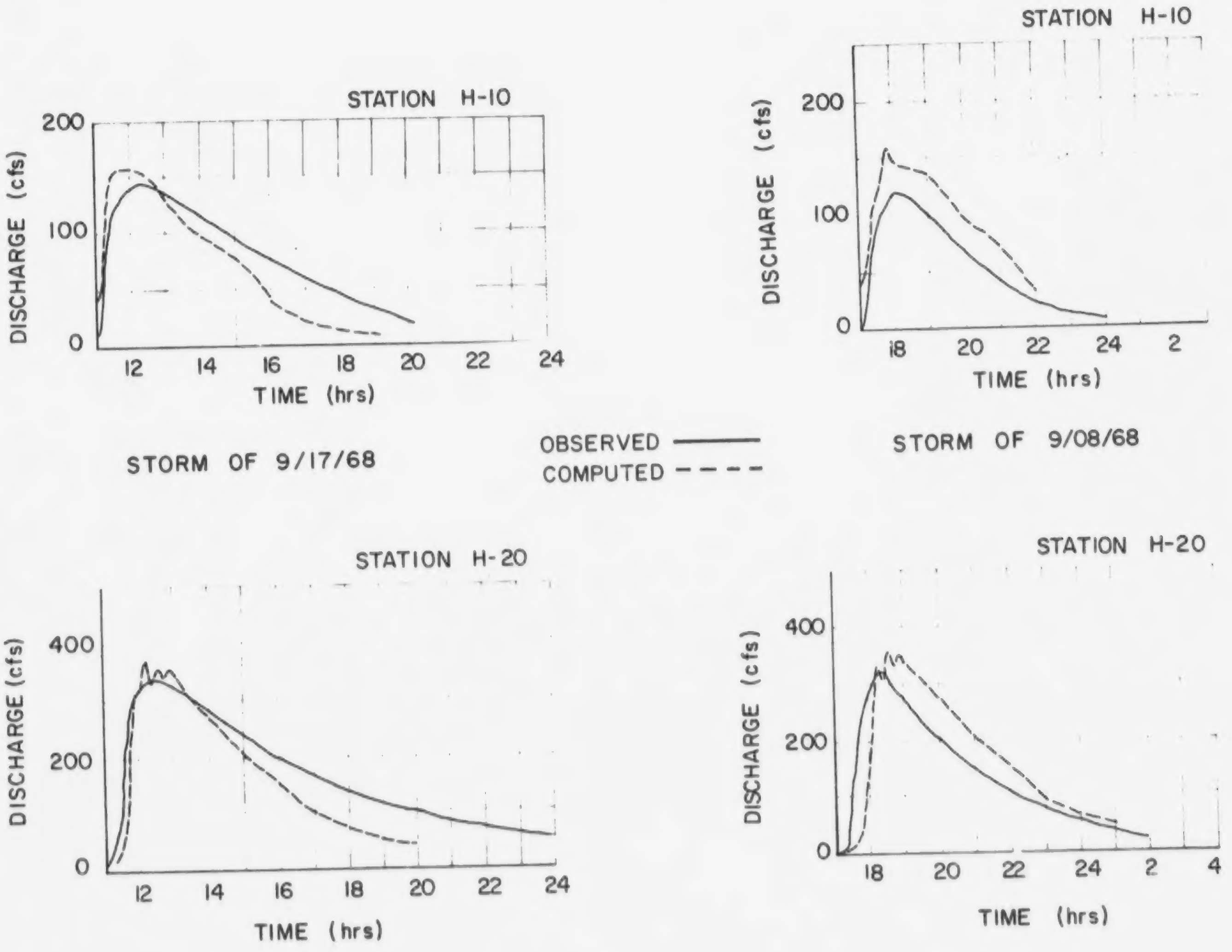


Fig. 38 Hydrographs at Stations H-10 and H-20

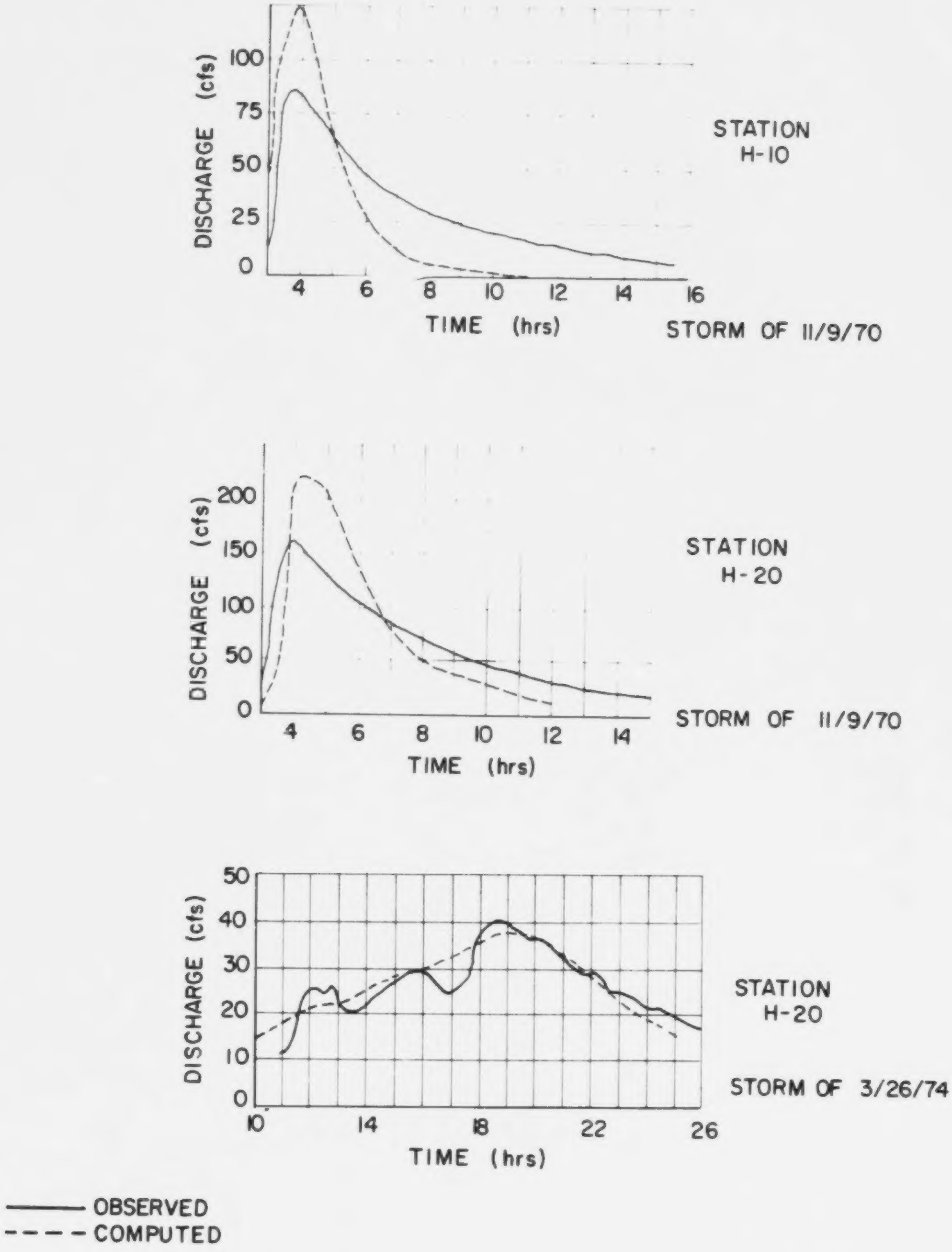


Fig. 39 Hydrographs at Stations H-10 and H-20

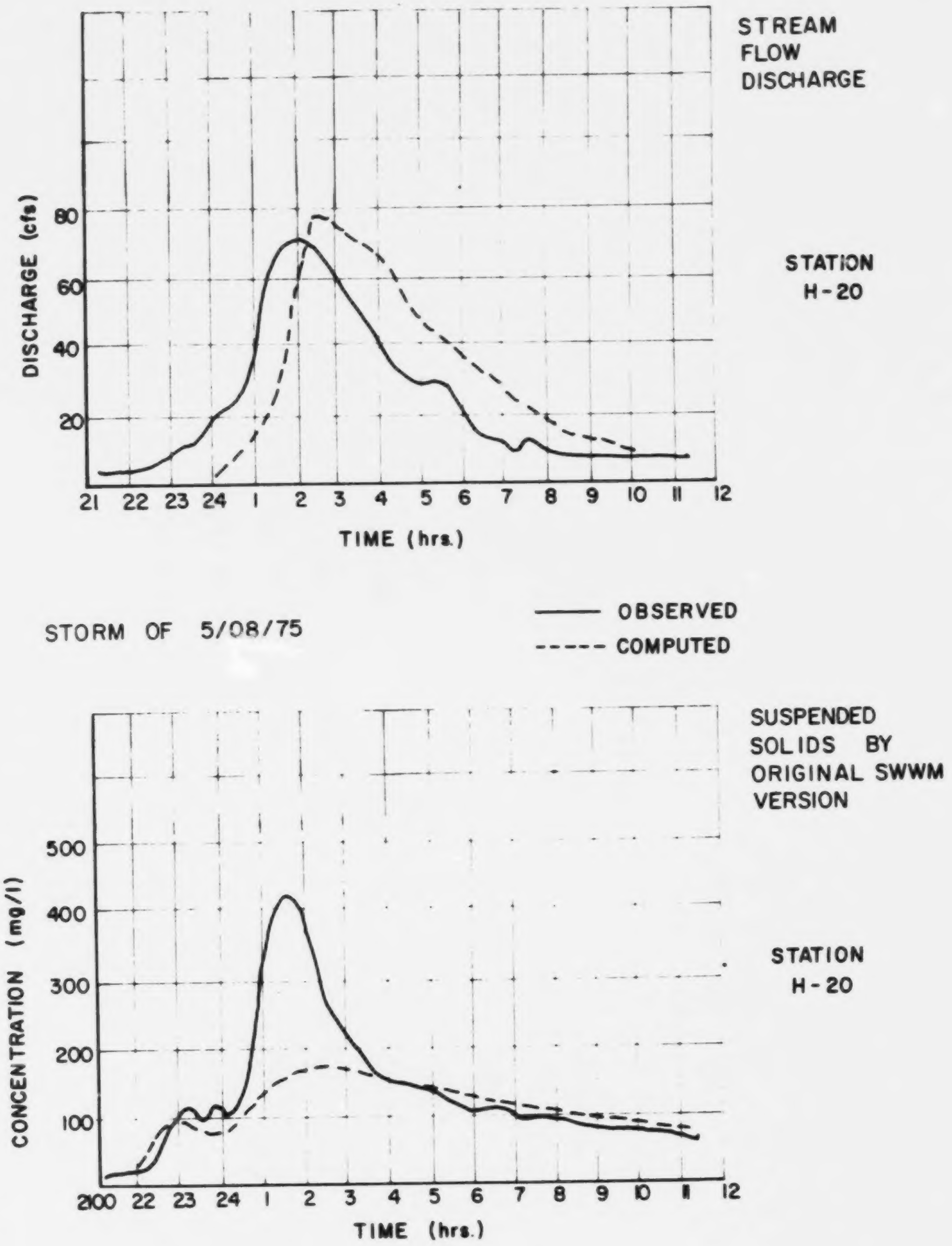


Fig. 40 Hydrographs and suspended solids concentrations at Station H-20

TABLE 21. HYDROGRAPH MODELING RESULTS FOR HUNTING BAYOU

STORM DATE	Location	RUNOFF VOLUME Observed	RUNOFF VOLUME Computed	PEAK FLOW Observed	PEAK FLOW Computed
9/8/68	H-10	1.43×10^6	1.85×10^6	121	160
	H-20	4.48×10^6	4.69×10^6	325	355
9/17/68	H-10	2.82×10^6	2.24×10^6	144	155
	H-20	8.37×10^6	6.02×10^6	333	365
11/9/70	H-10	1.5×10^6	0.97×10^6	85	125
	H-20	3.5×10^6	2.65×10^6	161	220
3/26/74	H-20	1.46×10^6	1.93×10^6	40	38
5/8/75	H-20	1.38×10^6	1.32×10^6	73	80

TABLE 22. HUNTING BAYOU - STORM OF 5/08/75, POLLUTANT LOADING RATES

Pollutant	Loading Rates (lb/ac) Residential	Construction	Undeveloped	Pollutant Removal Coefficient (Washoff exponent, k)
Pollutograph Reproduction				
Suspended Solids	1.850	2.310	0.230	21.2
Total COD	3.540	2.300	2.300	15.9
Nitrates	0.003	0.001	0.001	20.0
Total Phosphorous	0.008	0.003	0.002	20.0
Pollutant Mass Transport Rate Reproduction				
Suspended Solids	4.00	5.000	0.500	35.0
Total COD	7.800	5.100	5.100	30.0
Nitrates	0.007	0.003	0.003	20.0
Total Phosphorous	0.020	0.004	0.003	23.0

NOTE: (lb/ac) x (5.45) = (kg/ha)
Land uses are defined as follows:
Residential - areas with single and multi-family homes
Construction - areas with construction activity
Undeveloped - open undisturbed land

TABLE 23. WATER QUALITY MODELING RESULTS FOR HUNTING BAYOU - STORM OF 5/08/75

	Observed Data			Pollutograph Reproduction			Pollutant Mass Transport Rate Reproduction		
	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds $x10^2$	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds $x10^2$	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds $x10^2$
S. Solids	430.	114.	162.	431.	46.9	105.	1456	109.	231.
T. COD	630.	166.	140.	631.	66.7	161.	2478.	166.	374.
Nitrates	.66	.124	.40	.65	.05	.12	1.55	.125	.31
T. Phosphorous	1.68	.34	.84	1.76	.133	.32	5.01	.304	.75

NOTE: (lb/min) x (7.56) = (g/sec)
(lbs) x (2.2) = (kg)

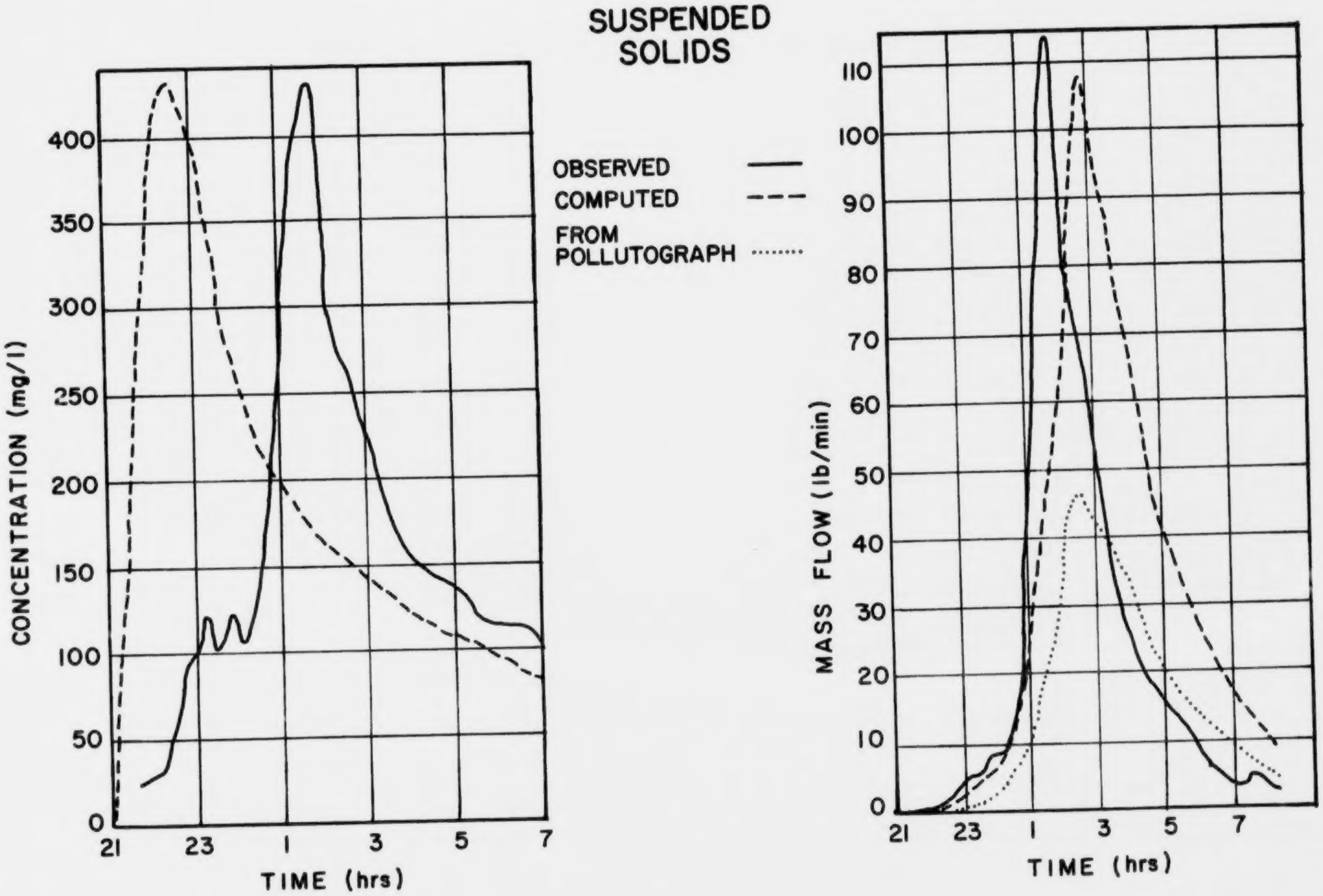
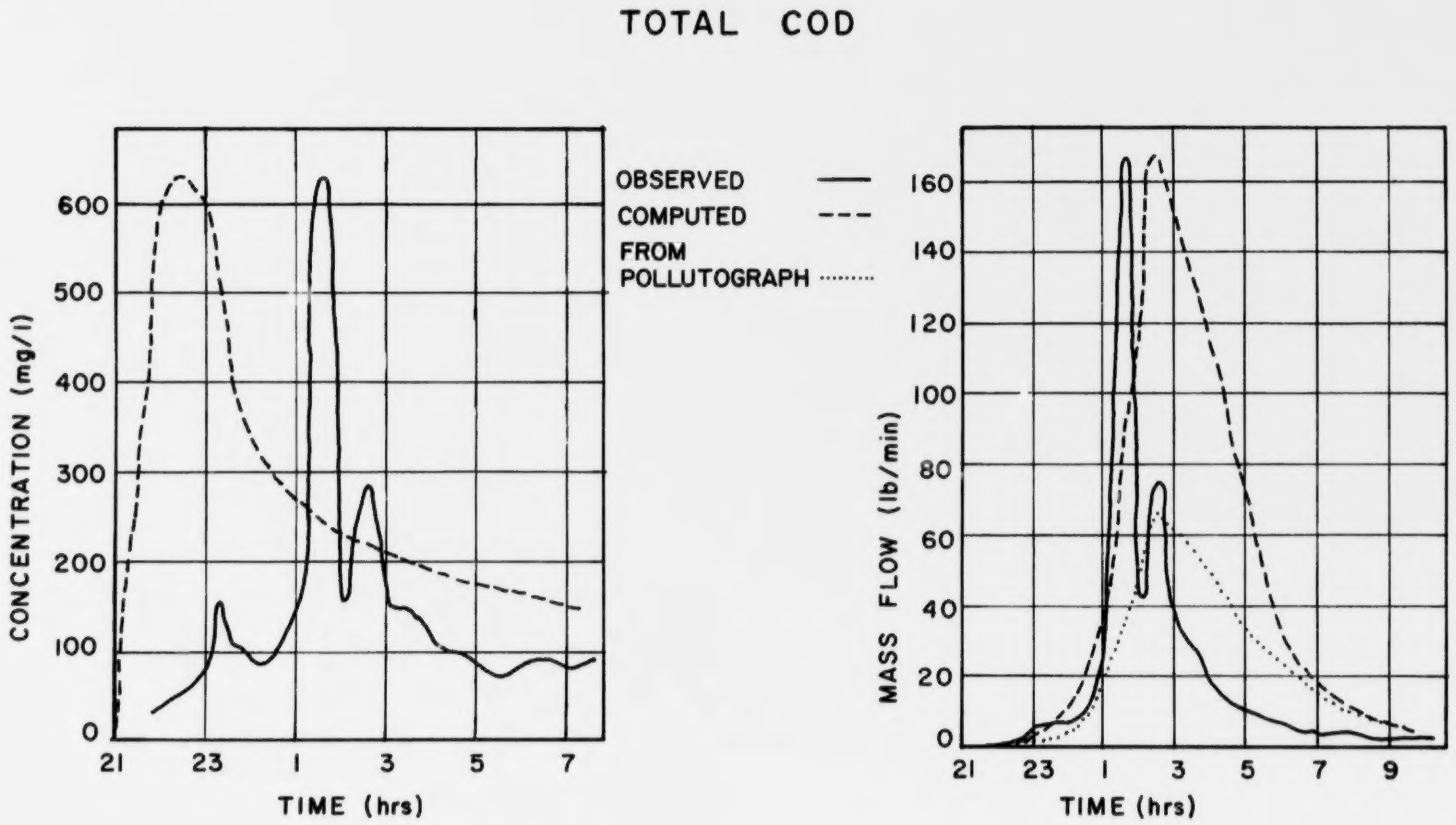
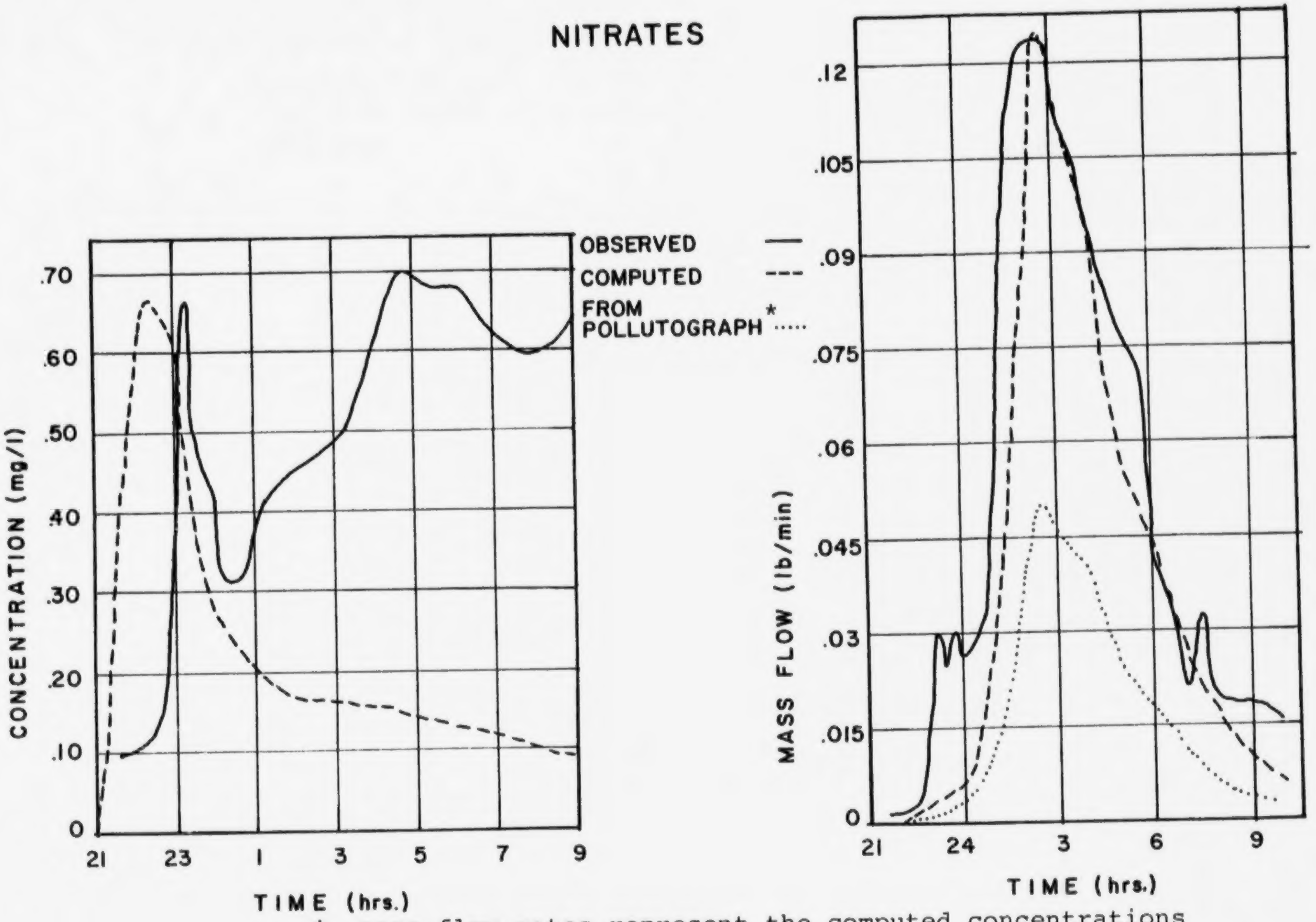


Fig. 41 Storm of 5/08/75 - Station H-20, suspended solids



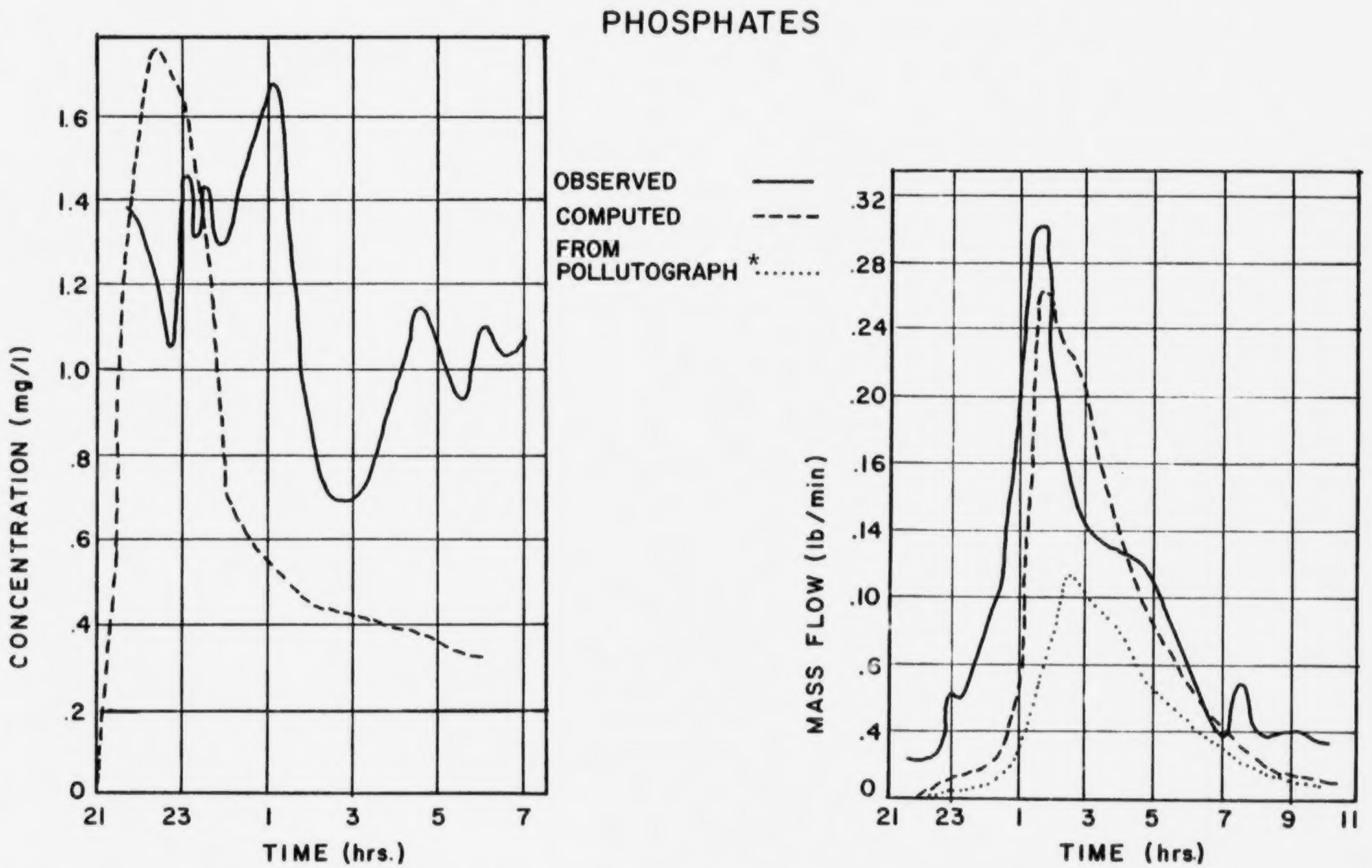
TOTAL COD

Fig. 42 Storm of 5/08/75 - Station H-20, COD



* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 43 Storm of 5/08/75 - Station H-20, nitrates



* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 44 Storm of 5/08/75 - Station H-20, phosphates

be determined. The predictions of total pounds of pollutant removed were improved 67% for nitrates and 75% for total phosphorus.

PANTHER BRANCH MODELING

The data input to the SWMM for the Panther Branch drainage system was developed from existing engineering maps and numerous site inspections of the watershed. Because of the low relief in topography, drainage area boundaries had to be visually determined in some areas.

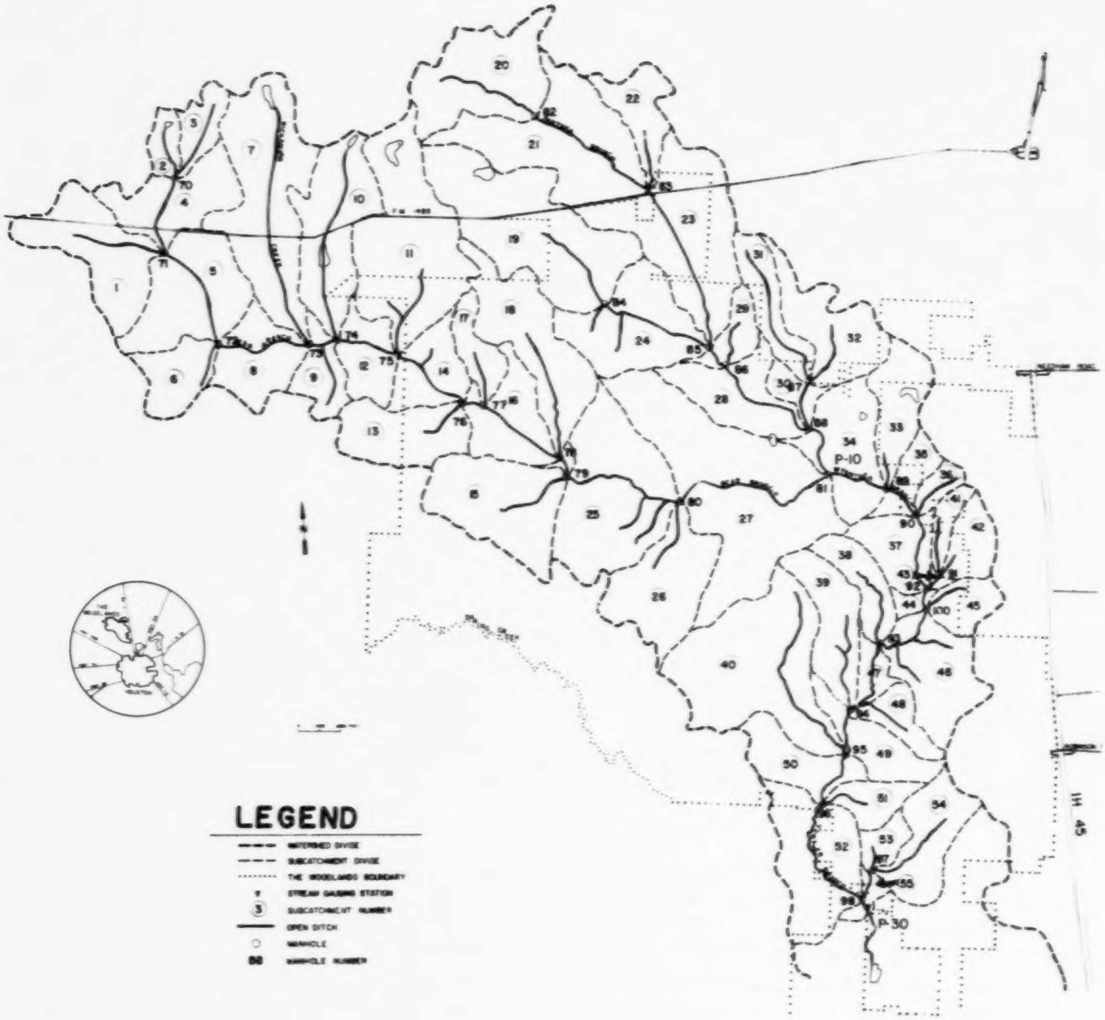
The modeled drainage system for Panther Branch is shown in Figure 45. The total drainage area of 874.40 hectares (21607 acres) is divided into 57 subcatchments ranging from 8.5 hectares (21 acres) to 552.80 hectares (1336 acres) as shown in Table 24. Other subcatchment data are also shown in Table 24. The input parameter called "width of subcatchment" is defined as the width over which overland flow occurs. Values for this parameter were first estimated by the method described in the SWMM User's Manual (2). These values were subsequently reduced by approximately 40% to achieve calibration. The land use classes for each subcatchment used for water quality modeling are listed in Table 25.

The Panther Branch drainage system is made up of 57 gutters whose characteristics are listed in Table 26, and 61 transport elements as described in Table 27.

Five storm events on Panther Branch have been modeled. Similar to the data for Hunting Bayou, all subcatchment, gutter and transport element data for Panther Branch were identical for all runs. The storm related data comprised of rainfall and infiltration coefficient data are listed in Tables 28 and 29, respectively. Infiltration rates were determined similar to those for Station H-20.

Observed and computed total flow volumes and peak flow rates for five storm events, which occured between 10/28/74 and 12/10/74, are compared in Table 30. Water quality data were available for the storms of October 28, 1974 and December 5,1974. The SWMM was used to model both water quantity and quality for these two storm events and only quantity of flow for the remaining three. Comparisons of observed to computed hydrographs are presented in Figures 46 and 47. The computed flow peaks and volumes agree well with the observed flows; the average absolute error in the volume of runoff was 14 percent of the observed peak. The temporal distribution of runoff between observed and computed hydrographs was good except for the storm events of October 28, 1974 and November 24, 1974, when the flow peaks between observed and computed hydrographs were approximately three hours apart.

An inspection in March 1975 revealed severe erosion at the area being cleared for Lake Woodlands on Panther Branch. A retaining wall that had been built to keep Panther Branch within



LEGEND
WATERSHED DIVIDE
SUBCATCHMENT DIVIDE
THE WOODLANDS BOUNDARY
STREAM GAUGING STATION
3 SUBCATCHMENT NUMBER
OPEN DITCH
MANHOLE
88 MANHOLE NUMBER
P-10
P-30
IH 45
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4
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Fig. 45 Subcatchments and drainage network - Panther Branch

TABLE 24. SUBCATCHMENT DATA, PANTHER BRANCH WATERSHED

Subcatchment No.	Width (ft)	Area (ac)	Percent Imperv.	Slope (ft/ft)	Resistance Factor		Surface Storage (in)		Infiltration Rate		
					Imperv.	Perv.	Imperv.	Perv.	(in/hr) Max.	Min.	(in/sec) Decay
1	4680.0	534.	1.0	.009	.200	.400	.005	.050	.50	.01	.00115
2	2100.0	99.	1.0	.006	.200	.400	.005	.050	.50	.01	.00115
3	3240.0	153.	1.0	.005	.200	.400	.005	.050	.50	.01	.00115
4	3540.0	405.	1.0	.009	.200	.400	.005	.050	.50	.01	.00115
5	5280.0	578.	1.0	.009	.200	.400	.005	.050	.50	.01	.00115
6	1620.0	243.	1.0	.006	.200	.400	.005	.050	.50	.01	.00115
7	7800.0	899.	1.0	.016	.200	.400	.005	.050	.50	.01	.00115
8	4380.0	338.	1.0	.014	.200	.400	.005	.050	.50	.01	.00115
9	2340.0	195.	1.0	.015	.200	.400	.005	.050	.50	.01	.00115
10	6660.0	584.	1.0	.015	.200	.400	.005	.050	.50	.01	.00115
11	4380.0	920.	1.0	.012	.200	.400	.005	.050	.50	.01	.00115
12	3600.0	248.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
13	2400.0	393.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
14	3780.0	275.	1.0	.016	.200	.400	.005	.050	.50	.01	.00115
15	4140.0	609.	1.0	.011	.200	.400	.005	.050	.50	.01	.00115
16	4680.0	353.	1.0	.018	.200	.400	.005	.050	.50	.01	.00115
17	1920.0	598.	1.0	.020	.200	.400	.005	.050	.50	.01	.00115
18	3780.0	598.	1.0	.015	.200	.400	.005	.050	.50	.01	.00115
19	4980.0	998.	1.0	.015	.200	.400	.005	.050	.50	.01	.00115
20	5280.0	612.	1.0	.015	.200	.400	.005	.050	.50	.01	.00115
21	6600.0	825.	1.0	.010	.200	.400	.005	.050	.50	01	.00115
22	2460.0	244.	1.0	.030	.200	.400	.005	.050	.50	.01	.00115
23	10560.0	851.	1.0	.018	.200	.400	.005	.050	.50	.01	.00115
24	6480.0	564.	1.0	.020	.200	.400	.005	.050	.50	.01	.00115
25	5820.0	870.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
26	2400.0	526.	1.0	.003	.200	.400	.005	.050	.50	.01	.00115
27	7320.0	1366.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
28	4200.0	390.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
29	2100.0	140.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
30	2280.0	101.	1.0	.012	.200	.400	.005	.050	.50	.01	.00115
31	1920.0	256.	1.0	.009	.200	.400	.005	.050	.50	.01	.00115
32	3420.0	421.	1.0	.012	.200	.400	.005	.050	.50	.01	.00115
33	2400.0	148.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
34	900.0	26.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
35	1080.0	301.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
36	3900.0	297.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
37	1800.0	151.	1.0	.011	.200	.400	.005	.050	.50	.01	.00115
38	1980.0	81.	1.0	.015	.200	.400	.005	.050	.50	.01	.00115
39	3780.0	257.	1.0	.011	.200	.400	.005	.050	.50	.01	.00115
40	3300.0	214.	1.0	.022	.200	.400	.005	.050	.50	.01	.00115
41	2640.0	354.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
42	7620.0	998.	1.0	.018	.200	.400	.005	.050	.50	.01	.00115
43	1800.0	96.	1.0	.012	.200	.400	.005	.050	.50	.01	.00115
44	1080.0	175.	1.0	.012	.200	.400	.005	.050	.50	.01	.00115
45	540.0	21.	1.0	.012	.200	.400	.005	.050	.50	.01	.00115
46	840.0	39.	1.0	.008	.200	.400	.005	.050	.50	.01	.00115
47	2400.0	169.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
48	3600.0	448.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115
49	2460.0	99.	1.0	.025	.200	.400	.005	.050	.50	.01	.00115
50	2040.0	162.	1.0	.019	.200	.400	.005	.050	.50	.01	.00115
51	1800.0	266.	1.0	.010	.200	.400	.005	.050	.50	.01	.00115
52	2040.0	281.	1.0	.011	.200	.400	.005	.050	.50	.01	.00115
53	1980.0	158.	1.0	.028	.200	.400	.005	.050	.50	.01	.00115
54	12500.0	584.	1.0	.020	.200	.400	.005	.050	.50	.01	.00115
55	2400.0	99.	1.0	.040	.200	.400	.005	.050	.50	.01	.00115
56	3780.0	374.	3.0	.012	.200	.400	.005	.050	.50	.01	.00115
57	1300.0	61.	1.0	.013	.200	.400	.005	.050	.50	.01	.00115

Total Tributary Area (acres), 21606.70

TABLE 25. LAND USE DATA, PANTHER BRANCH WATERSHED

Subarea Number	Land Use Class.	Total Gutter Length (100 ft)	Number of Catchbasins
1	5	50.00	0.00
2	5	30.00	0.00
3	5	45.00	0.00
4	5	40.00	0.00
5	5	55.00	0.00
6	5	22.50	0.00
7	5	130.00	0.00
8	5	47.50	0.00
9	5	35.00	0.00
10	5	100.00	0.00
11	5	62.50	0.00
12	5	31.00	0.00
13	5	25.00	0.00
14	5	41.00	0.00
15	5	40.00	0.00
16	5	46.00	0.00
17	5	27.50	0.00
18	5	70.00	0.00
19	5	50.00	0.00
20	5	55.00	0.00
21	5	71.00	0.00
22	5	30.00	0.00
23	5	85.00	0.00
24	5	60.00	0.00
25	5	62.50	0.00
26	5	40.00	0.00
27	5	82.50	0.00
28	5	57.50	0.00
29	5	22.50	0.00
30	5	22.50	0.00
31	5	80.00	0.00
32	5	35.00	0.00
33	5	24.00	0.00
34	5	10.00	0.00
35	5	20.00	0.00
36	5	60.00	0.00
37	5	20.00	0.00
38	5	30.00	0.00
39	5	40.00	0.00
40	5	42.50	0.00
41	5	50.00	0.00
42	5	97.50	0.00
43	5	30.00	0.00
44	5	15.00	0.00
45	5	10.00	0.00
46	5	10.00	0.00
47	5	15.00	0.00
48	5	35.00	0.00
49	5	40.00	0.00
50	5	35.00	0.00
51	5	22.50	0.00
52	5	30.00	0.00
53	5	25.00	0.00
54	5	62.50	0.00
55	5	15.00	0.00
56	1	50.00	0.00
57	5	20.00	0.00

Land Use Class is defined as follows:

1. Single and multi-family residential areas
5. Undeveloped urban open land

TABLE 26. GUTTER DATA, PANTHER BRANCH WATERSHED

Gutter Number	Width (ft)	Length (ft)	Slope (ft/ft)	Side Slopes		Manning	Overflow (in)
				L	R	n	
1	5.0	5000.	.003	6.0	6.0	.200	60.00
2	5.0	3000.	.002	6.0	6.0	.200	60.00
3	5.0	4500.	.003	6.0	6.0	.200	60.00
4	5.0	4000.	.004	6.0	6.0	.200	60.00
5	5.0	5500.	.002	6.0	6.0	.200	60.00
6	5.0	2250.	.009	6.0	6.0	.200	60.00
7	5.0	13000.	.003	6.0	6.0	.200	60.00
8	5.0	4750.	.003	6.0	6.0	.200	60.00
9	5.0	3500.	.007	6.0	6.0	.200	60.00
10	5.0	10000.	.005	6.0	6.0	.200	60.00
11	5.0	6250.	.003	6.0	6.0	.200	60.00
12	5.0	3100.	.003	6.0	6.0	.200	60.00
13	5.0	2500.	.009	6.0	6.0	.200	60.00
14	5.0	4100.	.001	6.0	6.0	.200	60.00
15	5.0	4000.	.003	6.0	6.0	.200	60.00
16	5.0	4600.	.002	6.0	6.0	.200	60.00
17	5.0	2750.	.006	6.0	6.0	.200	60.00
18	5.0	7000.	.004	6.0	6.0	.200	60.00
19	5.0	5000.	.003	6.0	6.0	.200	60.00
20	5.0	5500.	.005	6.0	6.0	.200	60.00
21	5.0	7100.	.002	6.0	6.0	.200	60.00
22	5.0	3000.	.003	6.0	6.0	.200	60.00
23	5.0	8500.	.002	6.0	6.0	.200	60.00
24	5.0	6000.	.003	6.0	6.0	.200	60.00
25	5.0	6250.	.001	6.0	6.0	.200	60.00
26	5.0	4000.	.003	6.0	6.0	.200	60.00
27	5.0	8250.	.001	6.0	6.0	.200	60.00
28	5.0	5750.	.001	6.0	6.0	.200	60.00
29	5.0	2250.	.013	6.0	6.0	.200	60.00
30	5.0	2250.	.004	6.0	6.0	.200	60.00
31	5.0	8000.	.005	6.0	6.0	.200	60.00
32	5.0	3500.	.003	6.0	6.0	.200	60.00
33	5.0	2400.	.001	6.0	6.0	.200	60.00
34	5.0	1000.	.001	6.0	6.0	.200	60.00
35	5.0	2000.	.004	6.0	6.0	.200	60.00
36	6.0	2700.	.001	6.0	6.0	.200	60.00
37	6.0	2000.	.001	5.0	5.0	.200	60.00
38	5.0	3000.	.007	6.0	6.0	.200	60.00
39	5.0	4000.	.001	6.0	6.0	.200	60.00
40	5.0	4250.	.006	6.0	6.0	.200	60.00
41	5.0	5000.	.005	6.0	6.0	.200	60.00
42	5.0	9750.	.004	6.0	6.0	.200	60.00
43	5.0	3000.	.002	6.0	6.0	.200	60.00
44	5.0	1500.	.004	6.0	6.0	.200	60.00
45	6.0	1000.	.003	6.0	6.0	.200	60.00
46	6.0	1000.	.001	6.0	6.0	.200	60.00
47	5.0	1500.	.010	6.0	6.0	.200	60.00
48	7.0	3500.	.001	6.0	6.0	.200	60.00
49	8.0	4000.	.001	6.0	6.0	.200	60.00
50	5.0	3500.	.007	6.0	6.0	.200	60.00
51	8.0	2250.	.001	6.0	6.0	.200	60.00
52	8.0	3000.	.001	6.0	6.0	.200	60.00
53	5.0	2500.	.006	6.0	6.0	.200	60.00
54	8.0	6250.	.001	6.0	6.0	.200	60.00
55	5.0	1500.	.003	6.0	6.0	.200	60.00
56	5.0	5000.	.005	6.0	6.0	.200	60.00
57	8.0	2000.	.001	6.0	6.0	.200	60.00

TABLE 27. TRANSPORT ELEMENT CHARACTERISTICS, PANTHER BRANCH WATERSHED

Ext.Elem Number	Description	Slope (ft/ft)	Distance (ft)	Manning n	Geom1	Geom2	Geom3	# of Barrels	AFull (sq ft)	QFull (cfs)	QMax (cfs)
4	Ditch	.0038	4000.0	.1800	6.0	-0.0	-0.0	1.0	554.062	478.268	478.268
5	Ditch	.0018	5500.0	.1800	6.0	-0.0	-0.0	1.0	554.062	329.167	329.167
8	Ditch	.0025	4750.0	.1800	6.0	-0.0	-0.0	1.0	554.062	387.927	387.927
9	Ditch	.0020	1500.0	.1800	6.0	-0.0	-0.0	1.0	554.062	346.972	346.972
12	Ditch	.0026	3100.0	.1800	6.0	-0.0	-0.0	1.0	554.062	395.609	395.609
14	Ditch	.0012	4100.0	.1800	8.0	-0.0	-0.0	1.0	984.998	578.816	578.816
161	Ditch	.0022	1000.0	.1800	8.0	-0.0	-0.0	1.0	984.998	783.721	763.721
16	Ditch	.0022	3600.0	.1800	9.0	-0.0	-0.0	1.0	1246.639	1072.922	1072.922
56	Ditch	.0008	1000.0	.1800	9.0	-0.0	-0.0	1.0	1246.639	646.996	646.996
25	Ditch	0008	6250.0	.1800	9.0	-0.0	-0.0	1.0	1246.639	646.996	646.996
27	Ditch	.0012	8250.0	.1800	9.0	-0.0	-0.0	1.0	1246.639	792.405	792.405
21	Ditch	.0023	7100.0	.1800	6.0	-0.0	-0.0	1.0	554.062	372.086	372.086
23	Ditch	.0018	8500.0	.1800	6.0	-0.0	-0.0	1.0	554.062	329.167	329.167
24	Ditch	.0028	6000.0	.1800	6.0	-0.0	-0.0	1.0	554.062	410.543	410.543
29	Ditch	.0014	1200.0	.1800	7.0	-0.0	-0.0	1.0	754.139	437.894	437.894
28	Ditch	.0014	5750.0	.1800	9.0	-0.0	-0.0	1.0	1246.639	855.895	855.895
30	Ditch	.0036	2250.0	.1800	6.0	-0.0	-0.0	1.0	554.062	465.512	465.512
34	Ditch	.0008	3000.0	.1800	9.0	-0.0	-0.0	1.0	1246.639	646.996	646.996
134	Ditch	.0008	3000.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	1927.987	1927.987
35	Ditch	.0009	2000.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	2044.939	2044.939
37	Ditch	.0006	4000.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	1669.686	1669.686
43	Ditch	.0030	1000.0	.0800	6.0	-0.0	-0.0	1.0	554.062	956.144	956.144
44	Ditch	.0006	1000.0	.0800	9.0	-0.0	-0.0	1.0	1246.639	1260.709	1260.709
46	Ditch	.0010	3500.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	2155.555	2155.555
47	Ditch	.0012	4000.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	2361.292	2361.292
49	Ditch	.0010	2250.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	2155.555	2155.555
50	Ditch	.0007	3000.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	1803.467	1803.467
52	Ditch	.0007	6250.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	1803.467	1803.467
551	Ditch	.0007	1100.0	.0800	6.0	-0.0	-0.0	1.0	554.062	461.861	461.861
55	Ditch	.0007	900.0	.0800	10.0	-0.0	-0.0	1.0	1539.060	1803.467	1803.467
70	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
71	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
72	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
73	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
74	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
75	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
76	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
77	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
78	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
79	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
80	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
81	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
82	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
83	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
84	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
85	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
86	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
87	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
88	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
89	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
90	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
91	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
92	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
100	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
93	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
94	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
95	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
96	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
97	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
98	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
99	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000

TABLE 28. RAINFALL DATA, PANTHER BRANCH WATERSHED

Date of Storm	11/10/74			12/10/74			11/24/74			12/5/74			10/28/74		
Time Interval, min.	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
Rain Gage No.	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
	.07	.03	.02	.04	.03	.00	.45	.45	1.00	.19	.29	.18	.00	.07	.00
	.07	.03	.02	.04	.03	.05	.45	.45	.20	.19	.29	.18	.00	.07	.05
	.07	.03	.03	.04	.04	.03	.10	.10	.20	.19	.29	.50	.00	.07	.05
	.07	.03	.03	.04	.05	.03	.10	.10	.04	.19	29	.20	.00	.02	.10
	.07	.33	.10	.05	.05	.05	.05	.05	.04	.00	.05	.20	.40	.02	.00
	.07	.33	.10	.05	.05	.00	.05	.05	.03	.05	.05	.05	.49	.02	.00
	.07	.00	.00	.65	.05	.10	.05	.05	.03	.00	.00	.35	.00	.10	.20
	.35	.19	.15	.07	.35	.15	.05	.05	.09	.00	.00	.00	.00	.10	.00
	.18	.32	.25	.07	.08	.25	.05	.05	.09	.00	.00	.00	.49	.10	.30
	.17	.32	.25	.06	.08	.07	.05	.05	.08	.00	.00	.00	.00	.15	.00
	.07	.06	.05	.02	.08	.07	.10	.10	.08	.00	.00	.00	.00	.15	.00
	.07	.12	.09	.02	.00	.07	.10	.10	.08	.00	.00	.00	.20	.50	.00
	.06	.12	.09	.05	.00	.00	.10	.10	.08	.00	.00	.00	.00	.00	.00
	.04	.12	.09	.05	.05	.00	.07	.07	.03	.00	.00	.00	.00	.00	.15
	.04	.11	.09	.05	.05	.10	.07	.07	.03	.00	.00	.00	.00	.00	.35
	.03	.11	.09		.05	.10	.07	.07	.03	.00	.00	.00	.50	.35	.00
	.03	.00	.00				.07	.07	.03	.00	.00	.00	.00	.35	.00
							.07	.07	.03	.00	.00	.00	.00	.00	.00
							.07	.07	.03	.00	.00	.00	.00	.00	.00
							.02	.02	.03	.00	.00	.00	.00	.00	.00
							.02	.02	.03	.00	.00	.00	.00	.00	.00
							.01	.01		.00	.00	.00	.00	.00	.00
										.00	.00	.00	.00	.00	.00
										.30	.00	.00	.00	.00	.00
										.15	.10	.10	.00	.00	.00
													.00	.00	.00
													.00	.00	.00
													.00		
													.00	.00	.00
													.00		
													.00	.00	.00
													.35	.00	.00
													.05	.65	.00
														.25	.00
Total Rainfall, in.	1.53	2.25	1.45	1.30	1.04	.66	2.17	2.17	2.23	1.26	1.36	1.76	2.48	2.97	1.20

TABLE 29. INFILTRATION PARAMETERS, PANTHER BRANCH WATERSHED

STORM DATE	STATION	INFILTRATION RATES Initial in/hr	Final in/hr	Decay /sec
10/28/74	P-10	3.5	0.01	.0005
	P-30	3.5	0.01	.0005
11/10/74	P-10	0.3	0.01	.00115
	P-30	0.3	0.01	.00115
11/24/74	P-10	2.0	0.01	.00115
	P-30	2.0	0.01	.00115
12/05/74	P-10	0.5	0.01	.00115
	P-30	0.5	0.01	.00115
12/10/74	P-10	0.2	0.01	.00115
	P-30	0.2	0.01	.00115

TABLE 30. HYDROGRAPH MODELING RESULTS, PANTHER BRANCH WATERSHED

Date of Storm		Total Runoff ($ft^3 \times 10^6$)		Peak Flow Rate (cfs)	
		Observed	Computed	Observed	Computed
10/28/74	P-10	24.40	29.03	342	360
	P-30	39.34	36.16	376	410
11/10/74	P-10	64.48	53.44	979	600
	P-30	72.87	73.61	897	705
11/24/74	P-10	52.24	57.72	680	645
	P-30	73.70	78.97	774	735
12/05/74	P-10	36.06	32.66	273	315
	P-30	45.52	48.55	329	370
12/10/74	P-10	44.42	33.61	464	380
	P-30	51.73	43.02	517	425

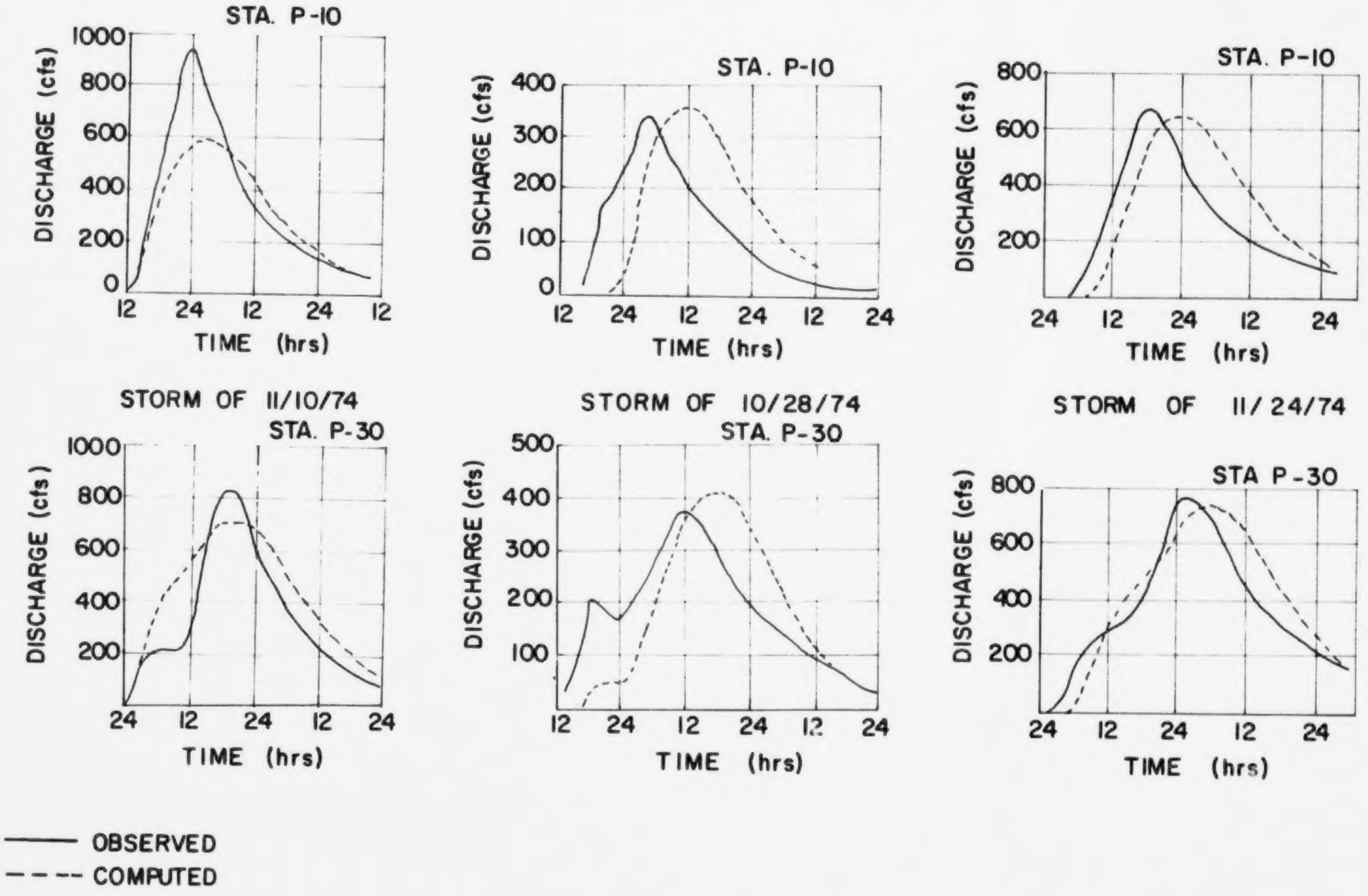


Fig. 46 Hydrographs at Stations P-10 and P-30

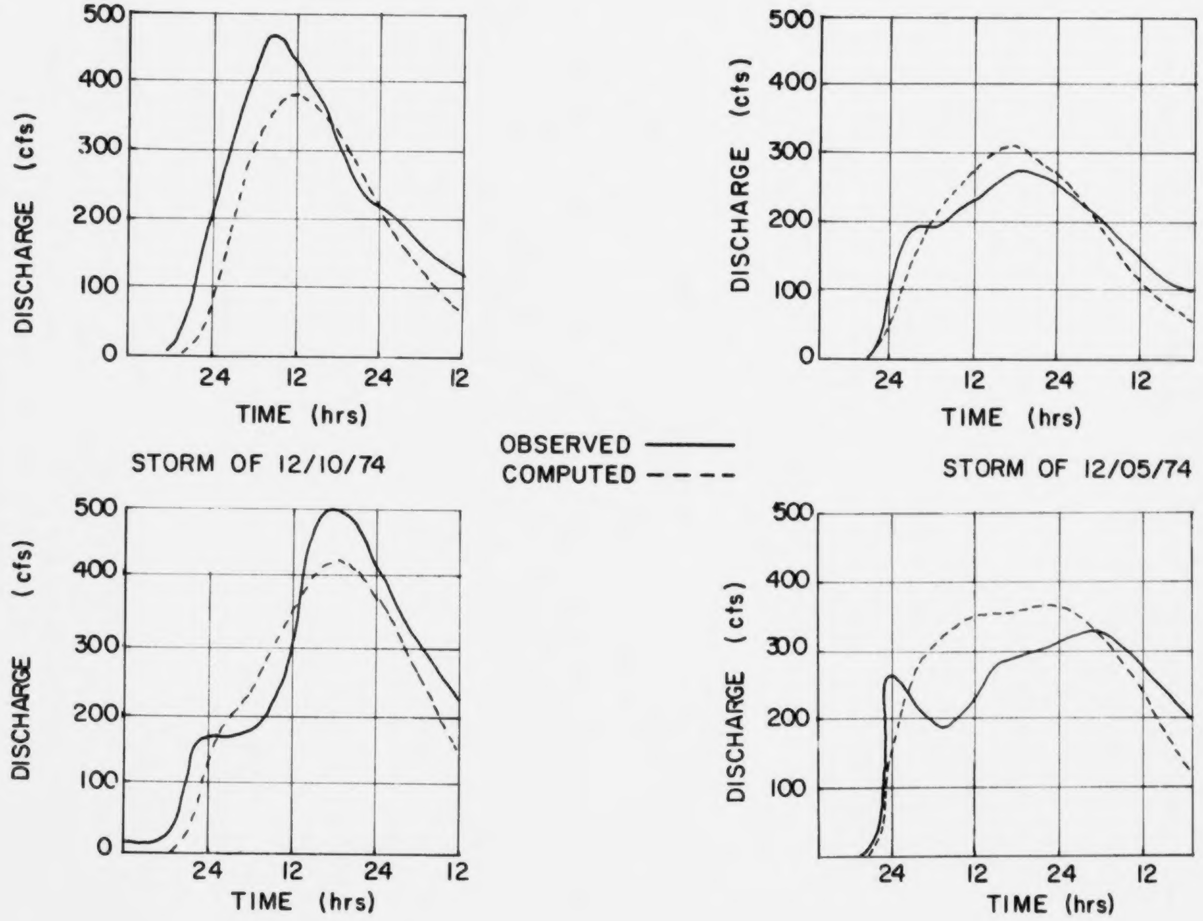


Fig. 47 Hydrographs at Stations P-10 and P-30

the original channel had been eroded away. This allowed Panther Branch to change its course and flow through the construction area which was being used as a sand pit. Downstream from the area there was heavy deposition of sand that had been washed away from the site. Erosion from consturction areas was the major source of suspended solids observed at P-30. During the study, peak concentrations of suspended solids declined at Station P-30. Further storm sampling may determine whether or not the erosion continues to be reduced.

A major drawback in applying the SWMM to The Woodlands is that the area below Station P-10 was in a transient state due to the development of Phase I. The continually changing land use affects the quality of runoff. For this reason the decision was made to regard Station P-10 as a control point. The area above this gage is in a relatively stable condition and will give a more accurate measurement of the pollutant loading due to the land use rather than from a construction area. Storm water runoff from a construction area can vary in quality from storm to storm depending on the stage of construction. Accounting for all construction areas and their erodibility prior to the storm event being modeled proved to be difficult. Consequently, it is presumed that several construction areas where the natural ground had been disturbed and stripped of the protective vegetative cover contributed more suspended solids than SWMM could predict from the available input data.

Investigation was also carried out on the time of occurrence and peak concentration of suspended solids. Except for the storms of April 11, 1974 and April 22, 1974, the observed suspended solids concentration peak occurred before the observed peak flow. The comparison would indicate that the peak concentration at Stations P-10 and P-30 occurred at a flow of 0.065 times the peak flow of the storm.

During the storm of October 28, 1974, measured at Station P-30, the rainfall caused two peaks in flow to occur, as shown in Figure 46. In modeling the second peak the original SWMM calculated a suspended solids peak concentration of 272.9 mg/l. Suspended solids for this storm had an observed peak of 1000 mg/l.

The December 5, 1974 storm had a modeling advantage in that it was the only storm analyzed where an upstream gage (P-10) and a downstream gage (P-30) had samples taken simultaneously. Since the entire drainage area had the same land use before development began, most differences between the upstream gage and the downstream gage can be attributed to the changing land use in the Phase I development area.

Using the original SWMM version, the computed peak concentration of suspended solids at Station P-10 was 142 mg/l compared with an observed value of 130 mg/l. This is a good agreement and also the time of peak concentration was the same. The falling limb of the observed pollutograph occurred too rapidly resulting in a difference of about 11,330 kilograms (25,000 pounds). This is about a 40% error.

Suspended solids production at Station P-30 was about three times greater than at Station P-10. As shown in Figure 48 the SWMM again calculated a low value for suspended solids. The SWMM was consistently low on the suspended solids concentration for the Phase I development area.

The EH&A modified water quality version of the SWMM was also used to model the 12/5/74 storm at Stations P-10 and P-30. The observed and computed pollutographs for suspended solids, COD, nitrates and phosphates are compared in Figures 49 through 52 for Station P-10 and in Figures 53 through 56 for Station P-30. As was done for Station H-20, after the pollutographs were adequately matched, the corresponding pollutant mass transport rates were computed and are also shown in Figures 49 through 56. Again the correlation between observed and computed pollutant mass transport rates from pollutographs reproduction was unsatisfactory. And so the pollutant mass transport rate predictions were improved until the reproductions were acceptable. The loading rates used are shown in Table 31 and the output results are summarized in Table 32.

At Station P-10, the optimized suspended solids pollutograph yielded very high pollutant mass transport rates (Figure 49) but the optimized mass flow rate follows the data except for a single peak. Suspended solids predictions at Station P-30 were more compatible to observed data with slight differences for occurrence of peak pollutant mass transport rates. This condition was also observed for COD at Station P-30 where the total pounds of COD were also predicted too high. Modeling of phosphates at both P-10 and P-30 proved to be difficult. The best approximations after several attempts are shown in Figures 52 and 56. The modeling of nitrates, especially at Station P-30 was not entirely satisfactory. It is believed that only parts of the entire Panther Branch watershed supply nitrates and therefore the model which predicts nitrates from throughout the watershed would have to be adjusted for this condition.

SWALE 8 MODELING

Existing drainage and planning maps were used to develop the input data for Swale 8. Site inspections to determine drainage area boundaries and extent of construction were conducted on a periodic basis. This watershed is in a transition stage. During the project term, the channel was enlarged and construction of Lake C was underway. Lakes A and B had already been filled.

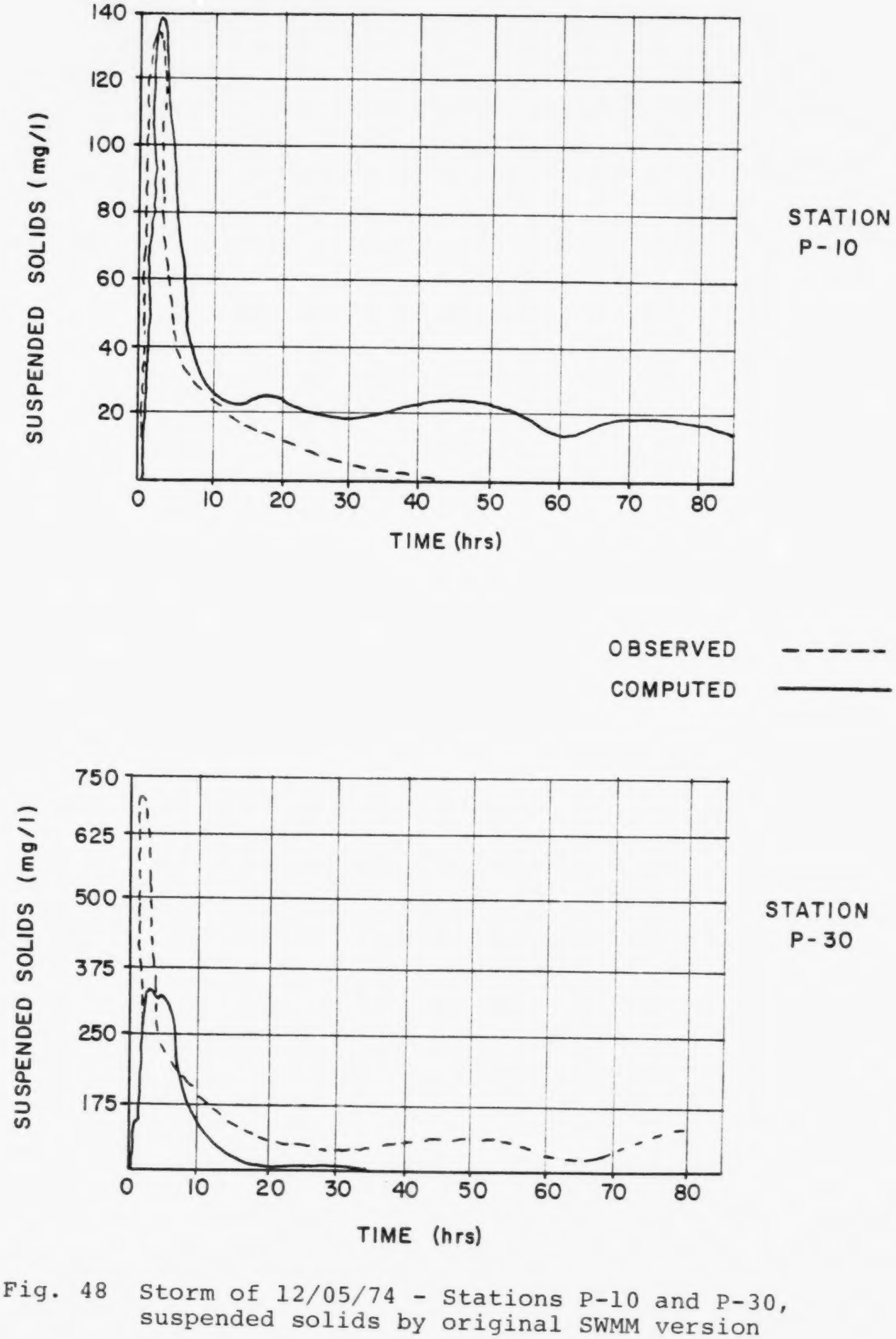
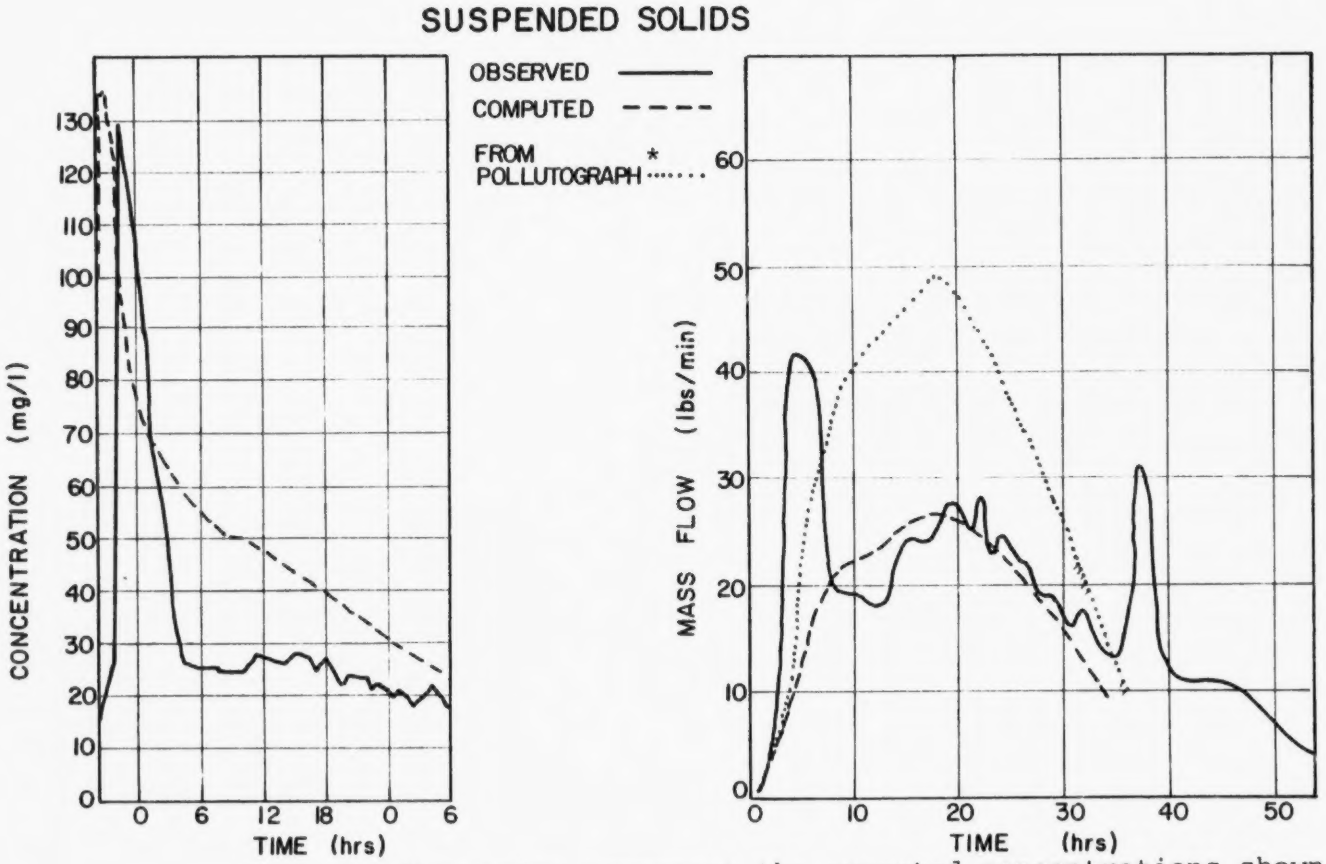
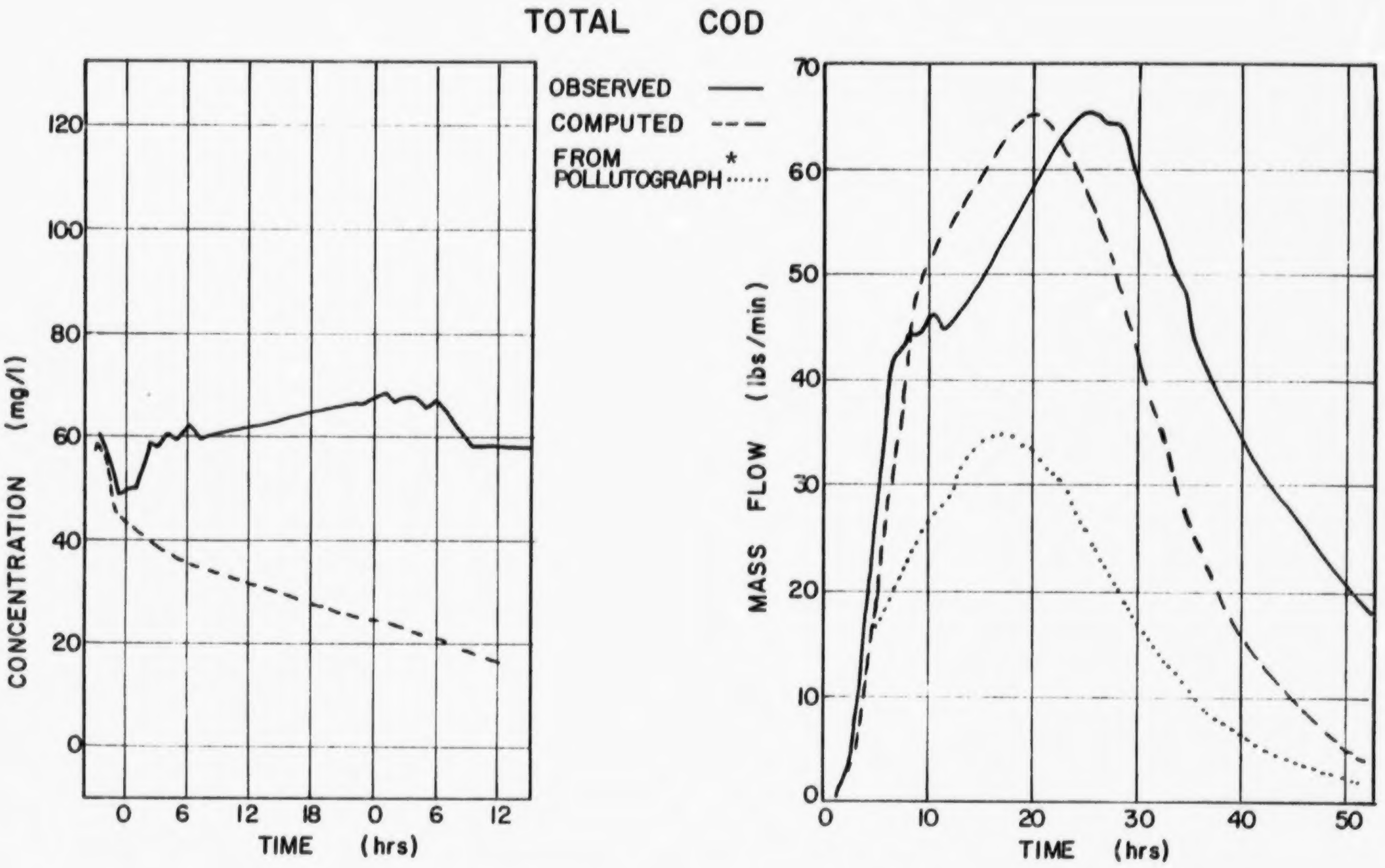


Fig. 48 Storm of 12/05/74 - Stations P-10 and P-30, suspended solids by original SWMM version



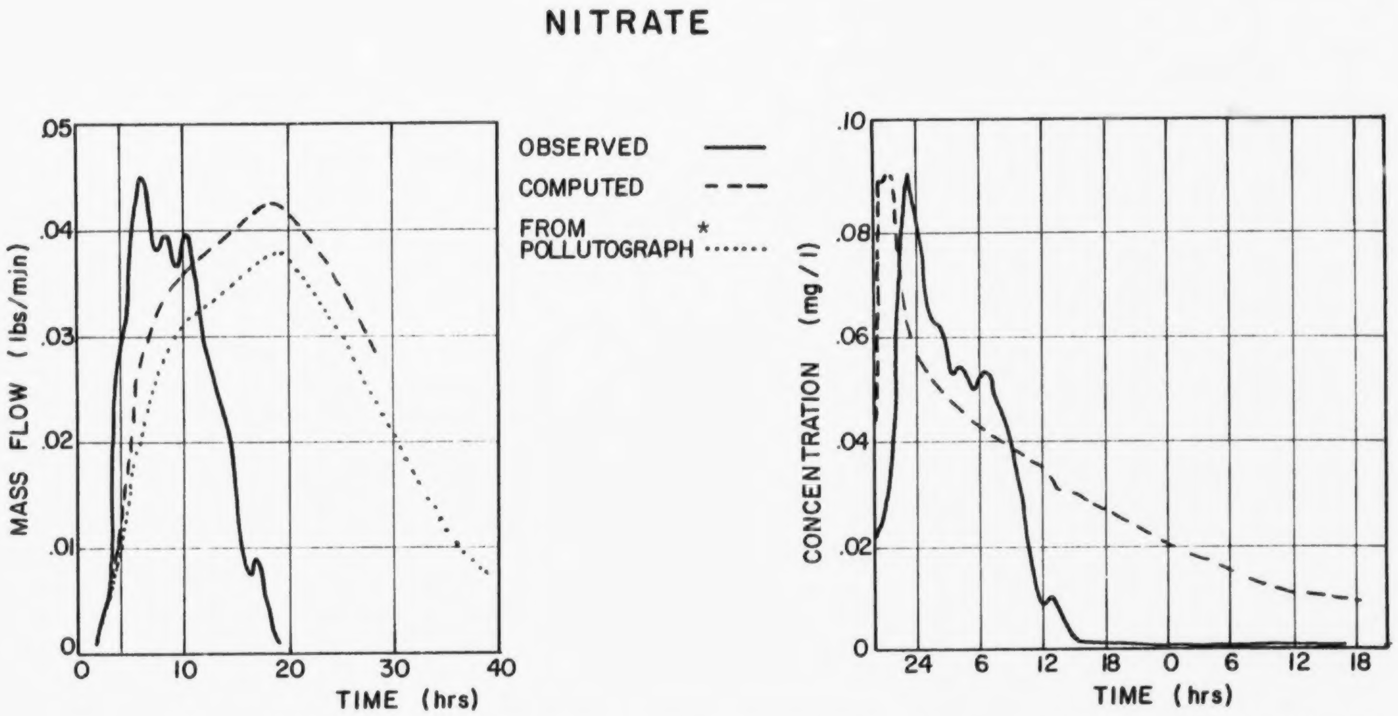
* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 49 Storm of 12/05/74 - Station P-10, suspended solids



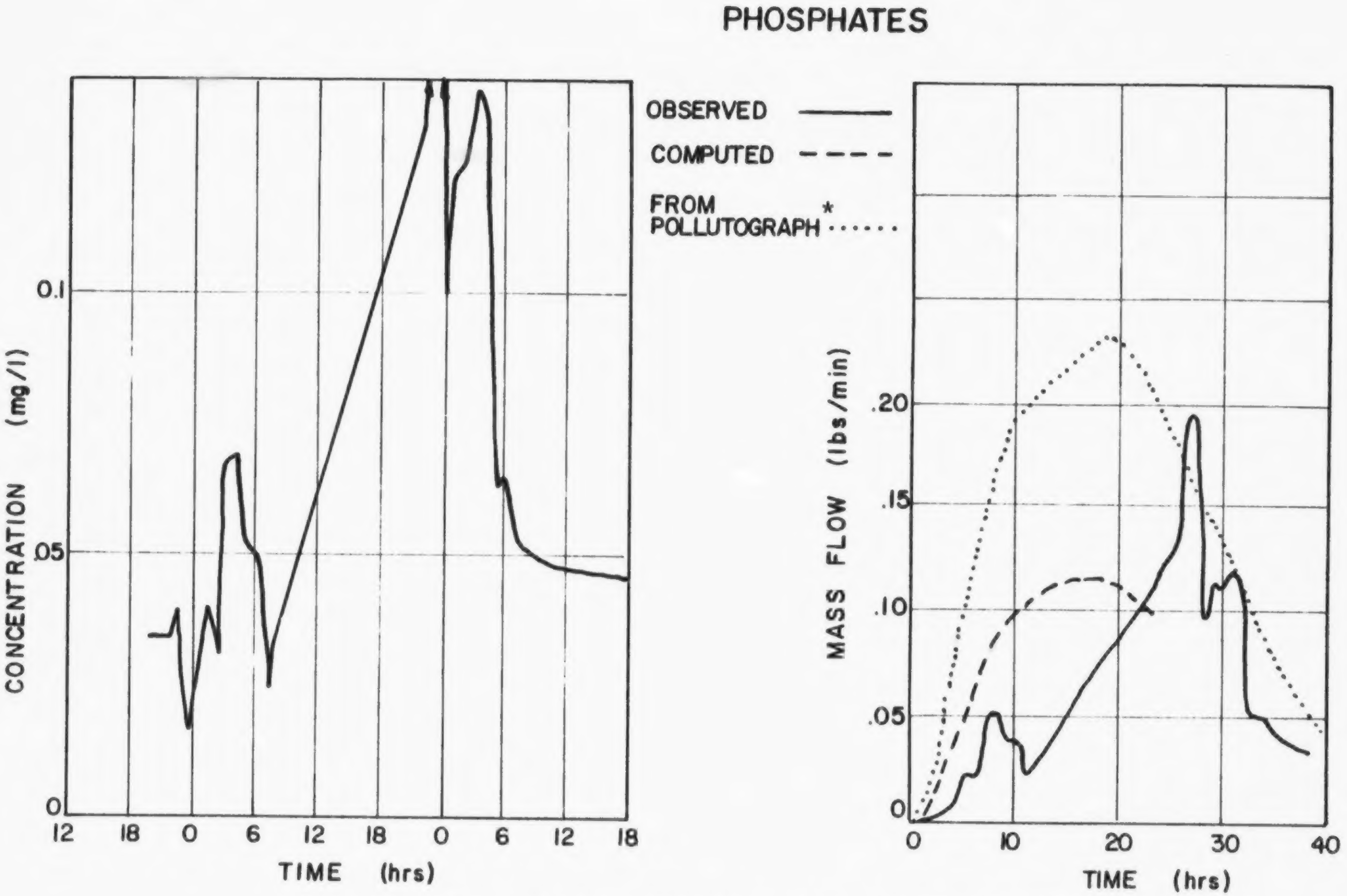
* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 50 Storm of 12/05/74 - Station P-10, COD



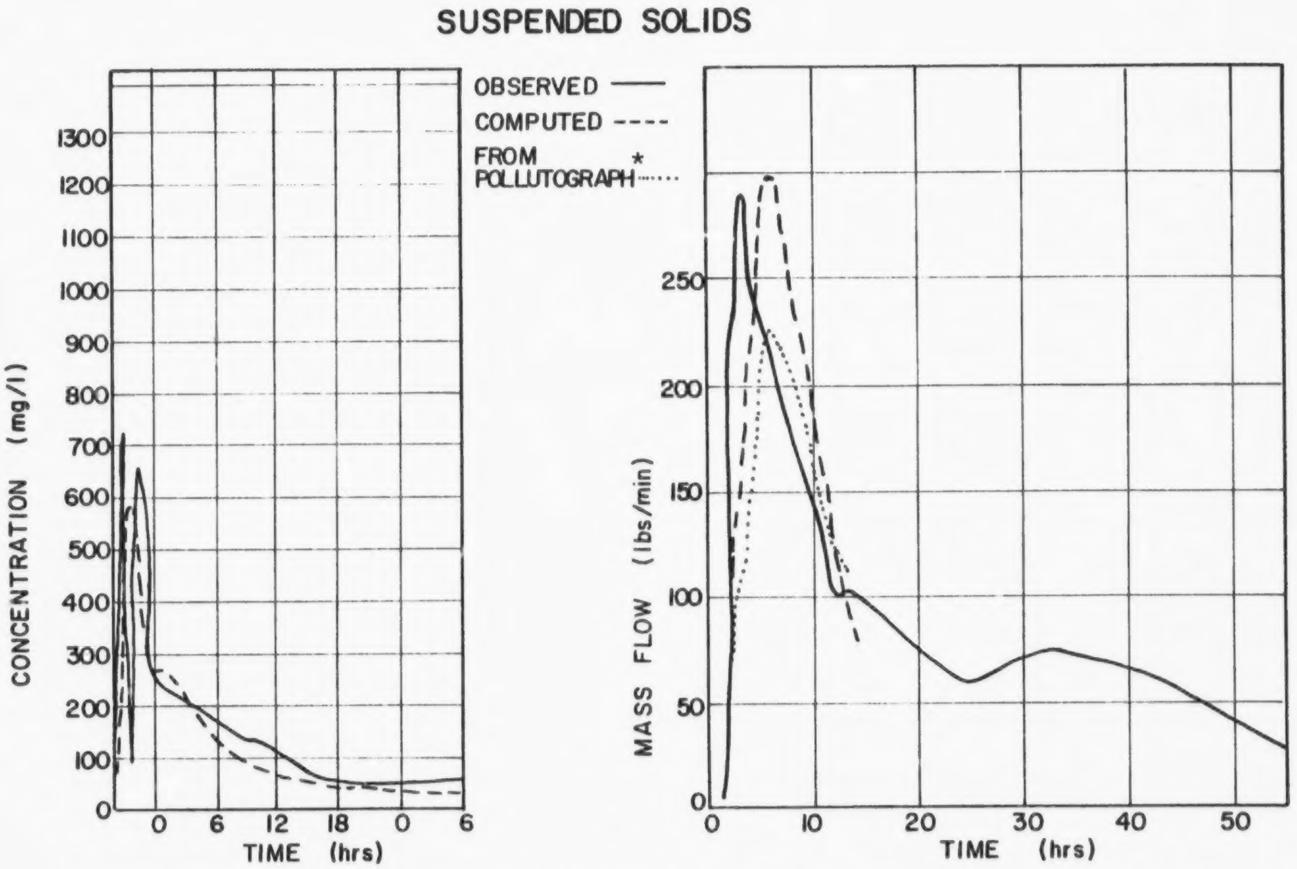
* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 51 Storm of 12/05/74 - Station P-10, Nitrates



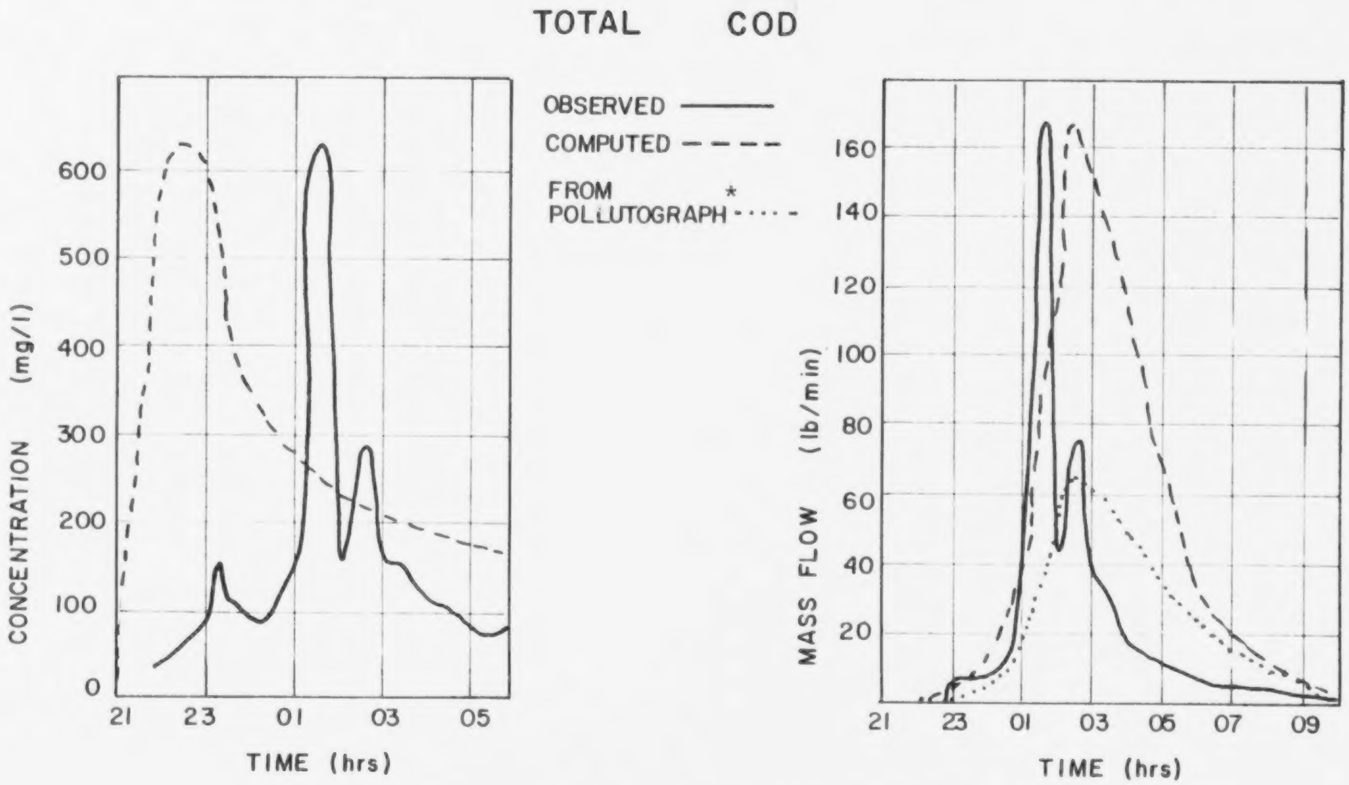
* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 52 Storm of 12/05/74 - Station P-10, Total Phosphorous



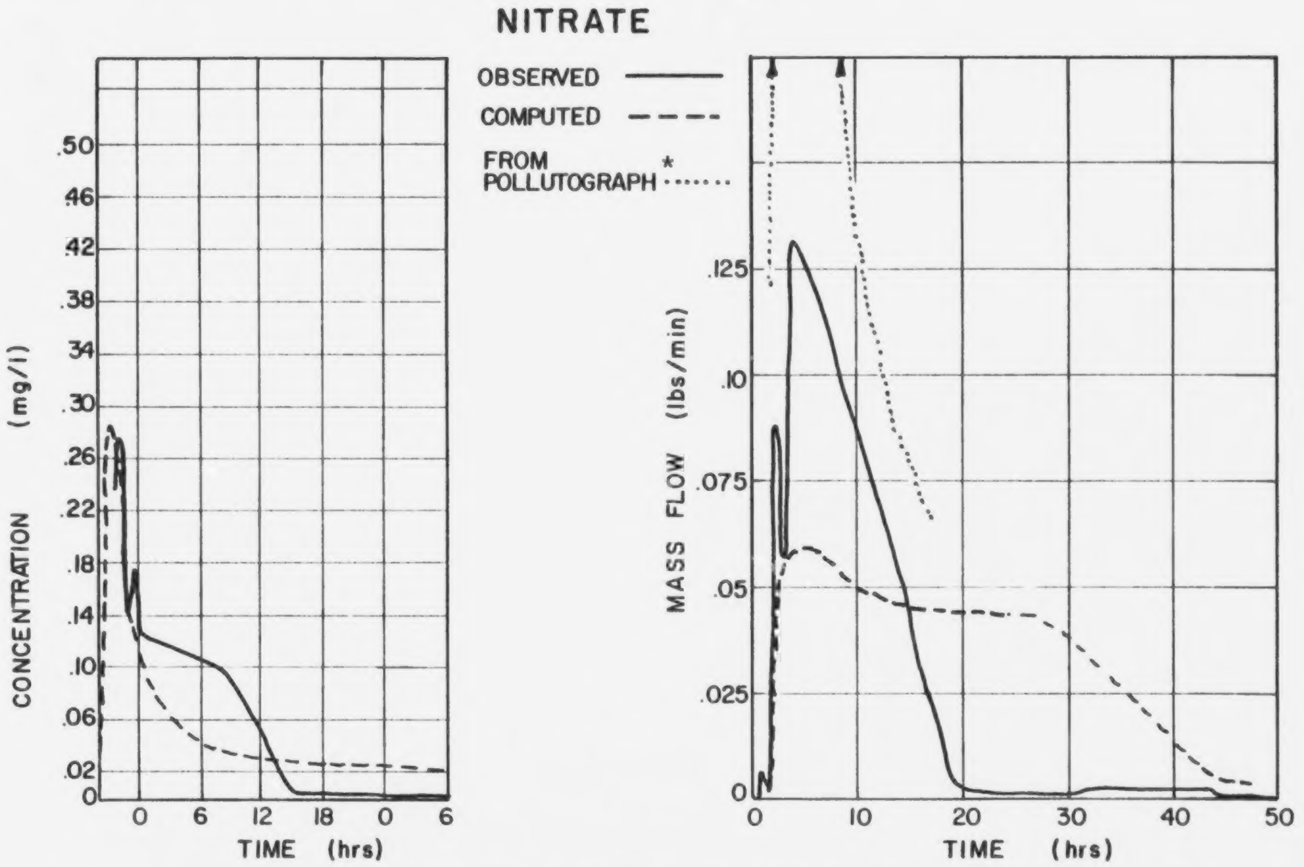
* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 53 Storm of 12/05/74 - Station P-30, Suspended Solids



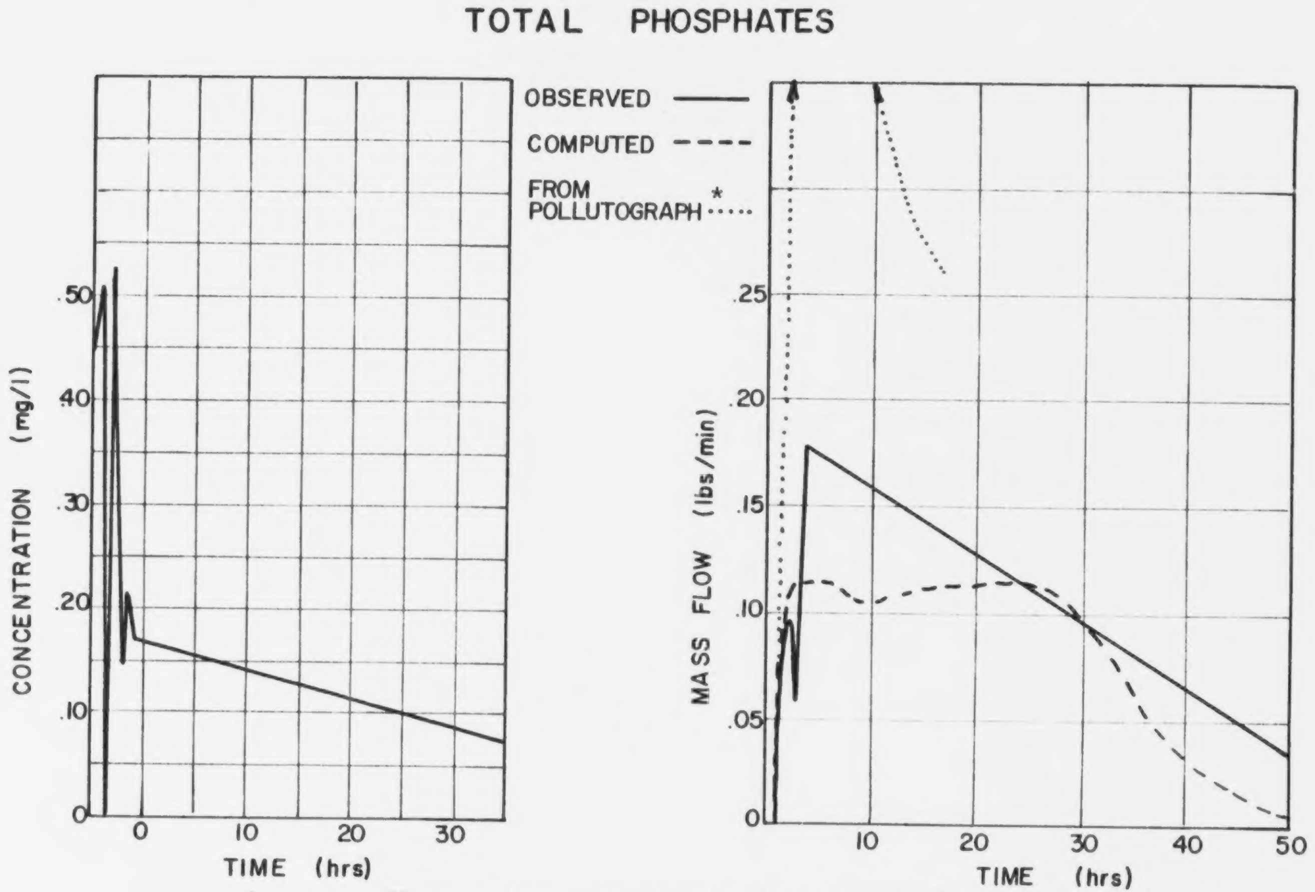
* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 54 Storm of 12/05/74 - Station P-30, COD



* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 55 Storm of 12/05/74 - Station P-30, Nitrates



* mass flow rates represent the computed concentrations shown on the left hand side of this figure

Fig. 56 Storm of 12/05/74 - Station P-30, Total Phosphates

TABLE 31. PANTHER BRANCH - STORM OF 12/05/74, POLLUTANT LOADING RATES

Pollutant	Loading Rates (lb/ac) Residential	Construction	Undeveloped	Pollutant Removal Coefficient (Washoff exponent, k)
	Pollutograph Reproduction			
Suspended Solids	25.0	35.0	4.0	7.0
COD	6.0	3.0	4.0	3.0
Nitrates	0.015	0.004	0.0035	5.3
Phosphates	0.17	0.03	0.02	5.3
	Pollutant Mass Transport Rate Reproduction			
Suspended Solids	35.0	45.0	2.2	7.0
COD	14.0	7.0	7.5	3.5
Nitrates	0.090	0.015	0.004	5.3
Phosphates	0.022	0.004	0.009	10.

NOTE: (lb/ac) x (1.12) = (kg/ha)
Land uses are defined as follows:
Residential - areas with single and multi-family homes
Construction - areas with construction activity
Undeveloped - open undisturbed land

TABLE 32. WATER QUALITY MODELING RESULTS FOR PANTHER BRANCH - STORM OF 12/05/74

	Observed Data			Pollutograph Reproduction			Pollutant Mass Transport Rate Reproduction		
	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds $x10^3$	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds $x10^3$	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds $x10^3$
STATION P-10									
SS	130.	27.		137.	48.		75.	27.	
COD	60.	66.		60.	35.		113.	66.	
Nitrates	.092	.045		.092	.037		.105	.043	
Phosphates	.20	.19		.52	.229		.433	.115	
STATION P-30									
SS	670.	290.	296	588.	233.	204.	814.	296.	205.
COD	63.	73.	196.	73.	35.	76.	169.	71.	149.
Nitrates	.276	.13	.08	.284	.068	.09	1.67	.38	.24
Phosphates	.53	.18	.33	3.2	.524	.72	.723	.114	.24

NOTE: (lb/min) x (7.56) = (g/sec)
(lbs) x (2.2) = (kg)

The drainage system for Swale 8 described in the SWMM is shown in Figure 57. The total drainage area of 185.87 hectares (459.3 acres) was divided into 10 subcatchments ranging from 9.30 hectares (23 acres) to 26.71 hectares (66 acres). All subcatchment data is listed in Table 33. Land uses for the upstream subcatchments were classified as open space, whereas the last three downstream subcatchments were designated as multi-family residential and commercial as shown in Table 34. Gutter data for all subcatchments are listed in Table 35. Seventeen drainage system elements, Table 36, were used to model the entire area. Of these, two elements were storage units, Lakes A and B, and all 6 channels were trapezoidal in shape as a result of channel enlargement. Table 36 lists all transport system element characteristics.

One storm event on Swale 8, that of 4/08/75, was modeled because the only other observed storm event, 3/13/75, had a peak inflow into Lake B of 0.06 cubic meters per second (2.0 cfs) from 2.06 centimeters (0.81 inches) of rainfall. The storm related temporal data for rainfall and infiltration are listed in Table 37.

The transitional phase of development in Swale 8 gave rise to several problems in modeling runoff. The most severe problem is the total lack of lake volume data. The topographic maps prior to lake construction show the natural ground contours, but the reservoir areas were used as borrow pits for fill material for the dams as well as other construction at The Woodlands. Consequently, the original storage capacity of the reservoirs was not known and no subsequent reservoir surveys have been conducted; therefore, the elevation-area-capacity data for these lakes was necessarily only approximate. Also groundwater was being pumped into the Lake A and B system and again the pumpage rate was not recorded.

A further complication arose from the fact that the outflow structure for Lake A is controlled by different outlets at different water surface elevations. The outflow rating curve (discharge as a function of water surface elevation) is composed of three segments, one controlled by the low flow orifice, the second controlled by weir flow through the flood discharge outlet which in turn is limited at extreme flows by the capacity of the outfall conduit and resulting in the third segment of the rating curve. The SWMM is not capable of modeling this complex outflow scheme.

Under the conditions described above, the modeling of runoff storage in the lakes proved to be difficult. Several attempts to model the outflow from Lake A (Station D-50) for the storm of 4/08/75 proved to be unsuccessful as shown in Figure 58.

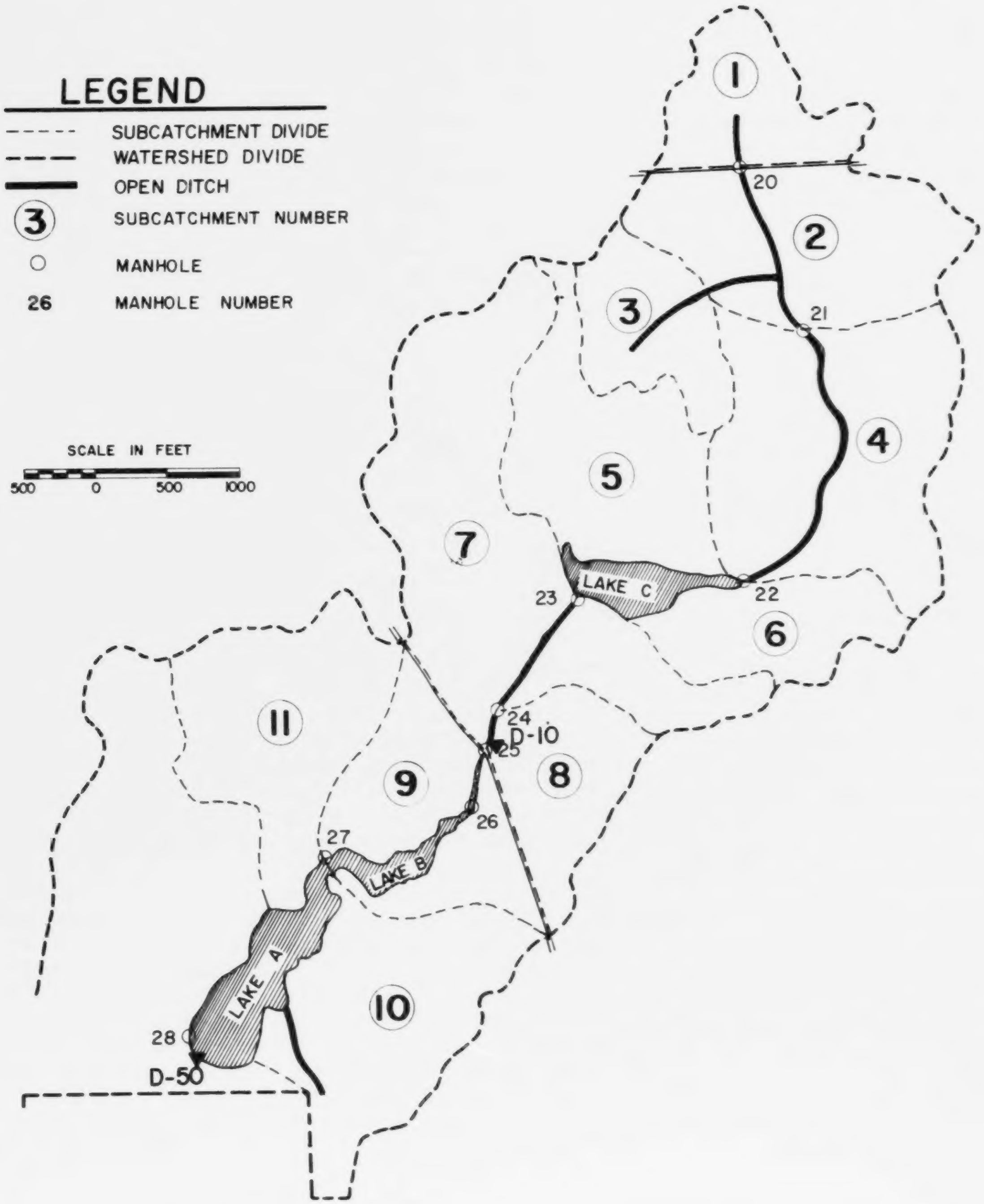

LEGEND
SUBCATCHMENT DIVIDE
WATERSHED DIVIDE
OPEN DITCH
3
SUBCATCHMENT NUMBER
MANHOLE
26
MANHOLE NUMBER
SCALE IN FEET
500
0
500
1000
1
2
3
4
5
6
7
8
9
10
11
20
21
22
23
24
25
26
27
28
LAKE C
LAKE B
LAKE A
D-10
D-50

Fig. 57 Subcatchments and drainage network - Swale 8

TABLE 33. SUBCATCHMENT DATA, SWALE 8 WATERSHED

Subcatch-ment No.	Width (ft)	Area (ac)	Percent Imperv.	Slope (ft/ft)	Resistance Factor Imperv.	Resistance Factor Perv.	Surface Storage (in) Imperv.	Surface Storage (in) Perv.	Infiltration Rate (in/hr) Max.	Infiltration Rate (in/hr) Min.	Infiltration Rate (in/sec) Decay
1	2600.0	23.	2.0	.007	.200	.400	.062	.184	2.00	.05	.00115
2	2100.0	47.	3.0	.003	.200	.400	.062	.184	2.00	.05	.00115
3	1150.0	23.	8.0	.004	.200	.400	.062	.184	2.00	.05	.00115
4	3700.0	66.	2.0	.008	.200	.400	.062	.184	2.00	.05	.00115
5	1100.0	44.	7.0	.005	.200	.400	.062	.184	2.00	.05	.00115
6	1500.0	28.	13.4	.015	.200	.400	.062	.184	2.00	.05	.00115
7	1300.0	81.	7.4	.008	.200	.400	.062	.184	2.00	.05	.00115
8	1700.0	39.	30.7	.020	.200	.400	.062	.184	2.00	.05	.00115
9	1300.0	55.	30.3	.020	.200	.400	.062	.184	2.00	.05	.00115
10	1200.0	52.	19.9	.015	.200	.400	.062	.184	2.00	.05	.00115

TABLE 34. LAND USE DATA, SWALE 8 WATERSHED

Subarea Number	Land Use Class.	Total Gutter Length (100 ft)	Number of Catchbasins
1	5	649.20	0.00
2	5	1320.80	0.00
3	5	646.40	0.00
4	5	1860.90	0.00
5	5	1222.90	0.00
6	5	778.00	0.00
7	5	1755.70	0.00
8	2	1100.00	0.00
9	2	1550.30	0.00
10	3	1455.20	0.00

NOTE: Land Use Class is defined as follows:
1. Single and multi-family residential areas
3. Business and commercial activity areas
5. Undeveloped urban open land

TABLE 35. GUTTER DATA, SWALE 8 WATERSHED

Gutter Number	Width (ft)	Length (ft)	Slope (ft/ft)	Side Slopes		Manning	Overflow (in)
				L	R	n	
1	4.0	1200.	.001	1.0	3.0	.030	36.00
2	3.0	500.	.005	6.0	6.0	.040	24.00
3	6.0	2200.	.001	1.0	3.0	.030	36.00
4	6.0	1000.	.005	2.0	3.0	.040	48.00

TABLE 36. TRANSPORT ELEMENT CHARACTERISTICS, SWALE 8 WATERSHED

Ext. Ele. Number	Description	Slope (ft/ft)	Distance (ft)	Manning n	Geom1 (ft)	Geom2 (ft)	Geom3 (ft)	# of Barrels	AFull (sq ft)	QFull (cfs)	QMax (cfs)
20	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
2	Trapezoid	.0010	1200.0	.0300	3.0	4.0	1.0	1.0	21.000	46.648	46.648
21	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
4	Trapezoid	.0010	2200.0	.0300	3.0	6.0	1.0	1.0	27.000	64.227	64.227
22	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
6	Trapezoid	.0005	1200.0	.0300	3.0	20.0	6.0	1.0	61.500	120.997	120.997
23	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
7	Trapezoid	.0050	1000.0	.0400	3.5	6.0	2.0	1.0	27.125	111.967	111.967
24	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
8	Trapezoid	.0050	300.0	.0400	4.0	6.0	2.0	1.0	32.000	140.027	140.027
25	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
9	Trapezoid	.0050	400.0	.0300	4.0	6.0	2.0	1.0	32.000	186.703	186.703
26	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
10	Stor.Unit	-0.000	-0.0	-0.000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
27	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
11	Stor.Unit	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000
28	Manhole	.1000	-0.0	.0130	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000

TABLE 37. RAINFALL AND INFILTRATION DATA, SWALE 8 WATERSHED

RAINFALL DATA
in inches at 20 minute intervals

.02	.02	.03	.05	.05
.06	.08	.00	.24	.80
.80	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.82	.12	.12	.12
.12	.12	.12	.12	.12

INFILTRATION RATES

Initial Rate, in/hr	0.75
Final Rate, in/hr	0.01
Decay Rate, /sec	0.00115

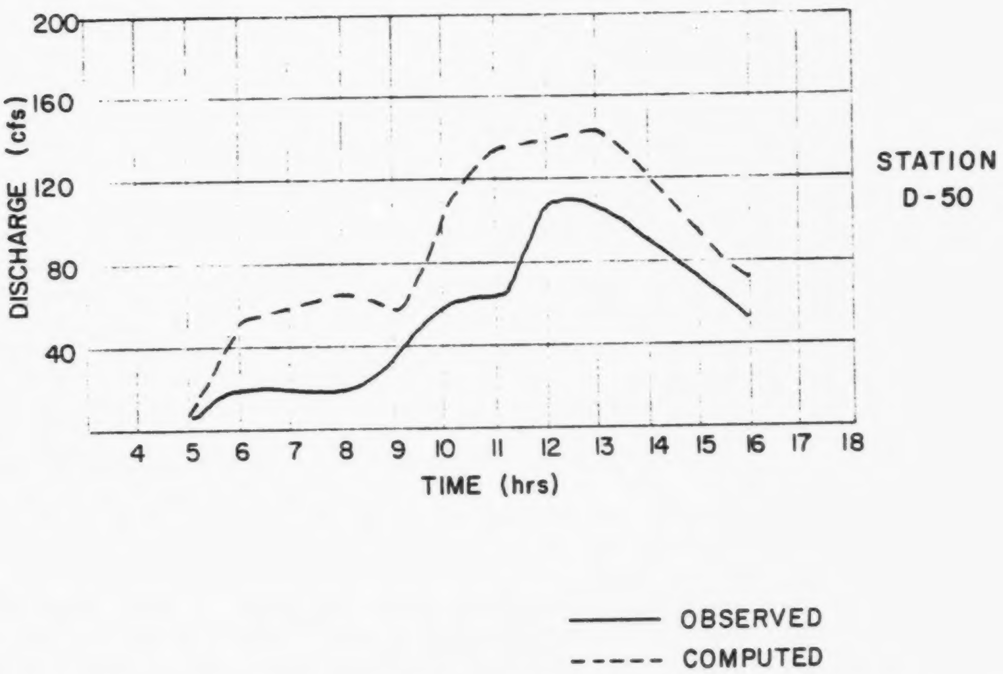


Fig. 58 Storm of 4/08/75 - Station D-50, hydrograph

The extent of assumed data was just too large in magnitude to even approximate the proper operation of Lakes A and B.

Consequently all further modeling was conducted only on that drainage area of Swale 8 upstream from Lake B (Station D-10). The results of this modeling effort are discussed in the following subsection.

EXISTING AND FUTURE DEVELOPMENT MODELING FOR SWALE 8

Due to various external influences, urban development at The Woodlands did not proceed as rapidly as had been expected. Consequently, site development plans were available for Phase I (Stage 1) only. In early 1976 a major portion of the Swale 8 watershed was being platted for development. Therefore all future development modeling was conducted for Swale 8.

The continued operation of Station D-10 promised to be beneficial in evaluating model predictions but unfortunately Lake C was constructed upstream of this gage. Data for Lake C has all the same problems associated with data for Lakes A and B. Therefore the model predictions were biased by approximate input data. Nevertheless, the SWMM was run for a phased development scheme for Swale 8. Three development scenarios were evaluated; existing conditions (development in Subcatchment 8 only and construction in Subcatchment 7), immediately developing conditions (development in Subcatchments 7 and 8 and construction in Subcatchments 3, 4, and 5), and future but not ultimate conditions (development in Subcatchments 3, 4, 5, 7 and 8 with no construction areas). Ultimate conditions would assume 100 percent urbanization and no development plans are available for these conditions.

Water quality predictions by both original and EH&A versions were attempted. Using plat maps provided by The Woodlands Development Corporation, the proposed urbanization area and curb lengths were measured. For Subcatchments 3,4, and 5 the proposed area to be urbanized amounted to 76, 73, and 72 percent, respectively. The average curb length per urbanized area was determined for these three subcatchments and applied to all other areas where curb lengths were not available. These computed and proposed curb lengths were used for pollutant generation by the original SWMM model. The changes in land use and increase in imperviousness were also computed and input to the SWMM. All of these data are listed in Table 38.

As described earlier, the EH&A quality prediction version required the input of loading rates for each pollutant. The curves shown in Figures 31 through 55 were used to derive each of the desired loading rates listed in Table 39. Land use and imperviousness data was the same as that for the original version (Table 38).

TABLE 38. LAND USE DATA FOR FUTURE DEVELOPMENT, SWALE 8

Sub-catchment	Total Area (Acres)	Urban Area (Acres)	Urban Area (%)	Curb Length (Feet)
1	23	0	0	2760
2	47	6	13	8480
3	23	17	76	3540
4	65	39	73	10720
5	41	30	72	14480
6	29	4	15	-
7	81	27	37	12440
8	25	14	56	4320

TABLE 39. SWALE 8 - STORM OF 4/08/75, POLLUTANT LOADING RATES

Pollutant	Loading Rates (lb/ac) Undeveloped	Residential	Construction	Pollutant Removal Coefficient (Washoff exponent, k)
	Pollutant Mass Transport Rate Reproduction			
Suspended Solids	31.	101.	123.	4.6
COD	3.3	6.2	3.1	3.0
Nitrates	.009	.016	.006	5.0
Phosphates	.0082	.020	.0037	10.0

NOTE: (lb/ac) x (1.12) = (kg/ha)
Land uses are defined as follows:
Residential - areas with single and multi-family homes
Construction - areas with construction activity
Undeveloped - open undisturbed land

Observed and computed hydrographs for the storm of 4/08/75 are compared in Figure 59. Observed and computed mass flow rates as determined by the original SWMM version for all three development conditions are compared in Figure 60.

As shown in Figure 59, the total hydrograph for the storm of 4/08/75 is composed of two hydrographs resulting from two distinct periods of rainfall separated by 3 hours and 20 minutes of no rainfall. Only the first period of rainfall, runoff and water quality was modeled. Based on previously described experience with pollutograph differences resulting from computed hydrographs, it was decided that only mass flow rates would be modeled.

Loading rates determined from the results of modeling at Station P-30 were used in the first run. It became evident that the initial loading rate estimates for developed areas were too low indicating the extreme effects of lake and golf course construction, as well as channel improvement. These activities were concentrated in the Swale 8 watershed and well diluted in the Panther Branch watershed; for example, the observed peak mass flow of suspended solids at Station D-10 was three times the peak mass flow computed from loading rates derived at Station P-30. Also, the areas already developed have not been stabilized - when rainfall intensities are sufficiently high, even the freshly sodded areas will erode severely; consequently, the loading rates for developed and construction areas in the Swale 8 watershed were much closer than expected. At Station D-10 the suspended solids loading rates from developed areas were 82% of the rate from construction areas. The same ratio at Station P-30 was 78%. The relatively high rates for developed areas indicate the severe erosion potential from recently developed areas. The results of modeling the storm of 4/08/75 for Swale 8 are shown in Figures 61 and 62 and in Table 40. Predictions based on loading rates determined for the 12/05/74 storm at Station P-30 are also presented.

The EH&A version of the SWMM was also run for the two development conditions described earlier. The storm of 4/08/75 was used to provide a basis for comparison between existing and future conditions. The pollutant mass transport rates predicted from future development are shown in Figures 61 through 66. As anticipated, the modeling of Subcatchment areas 3, 4 and 5 as construction areas changes the pollutant loads considerably; the changes range from an increase of 77% for suspended solids to a decrease of 8% for nitrates. After the construction phase of development has been completed, the peak pollutant loads do not decrease as may be expected, but the total pounds of pollutant does decrease. These dramatic environmental effects of construction activities are graphically illustrated in Figures 61 through 66 and listed in Table 41.

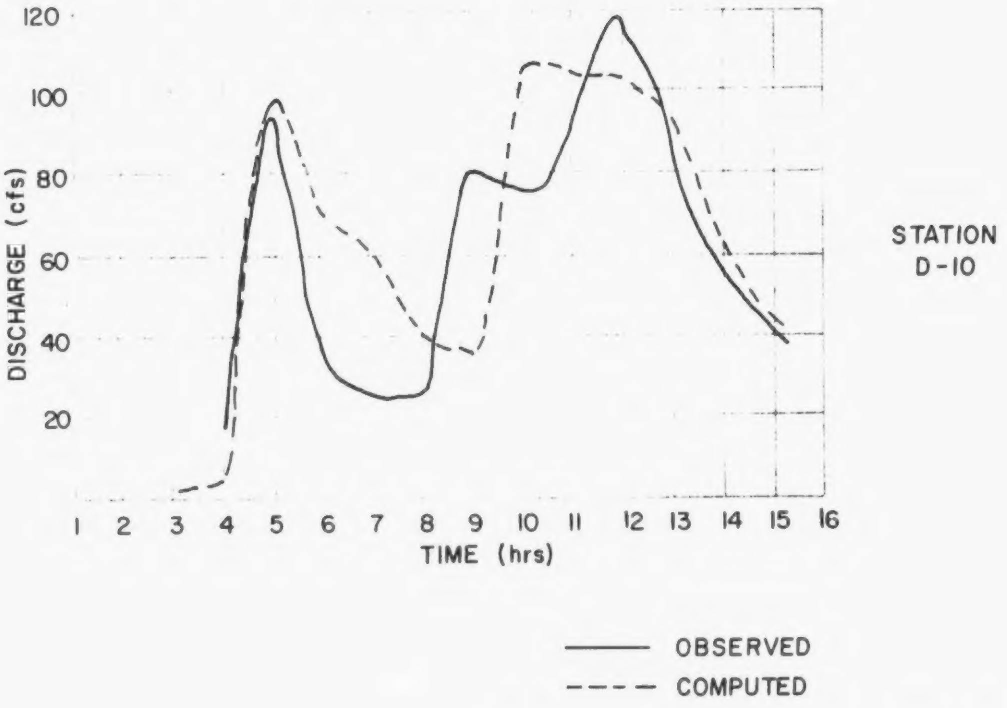


Fig. 59 Storm of 4/08/75 - Station D-10 Hydrograph

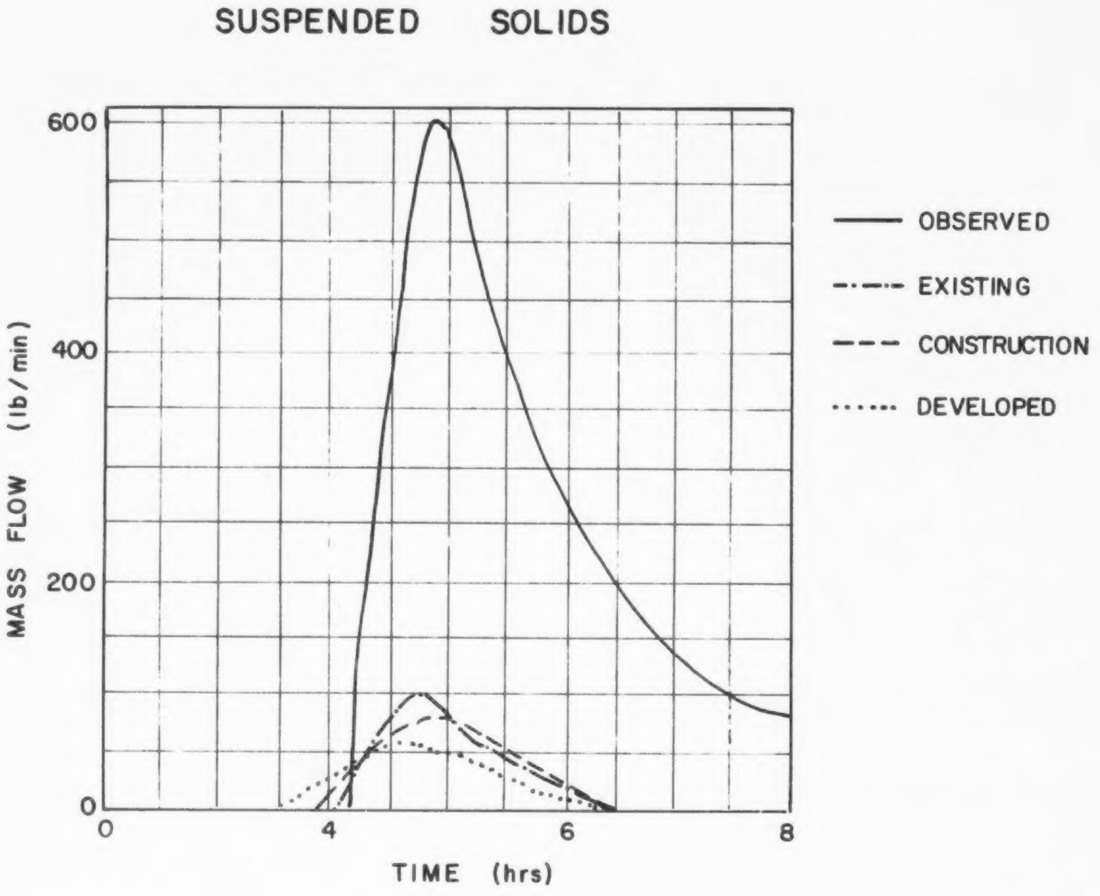


Fig. 60 Storm of 4/08/75 - Station D-10, Suspended Solids by Original SWMM Version

TABLE 40. WATER QUALITY MODELING RESULTS FOR STATION D-10 - STORM OF 4/08/75

	Observed Data			Pollutant Mass Transport Rate Reproduction		
	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds
Suspended Solids	2152.	608.	1891.	6348.	609.	65290
COD	87.	31.5	248.	217.	31.3	3539
Nitrates	2.105	.113	.73	.93	.108	9
Phosphates	.359	.130	.21	2.32	.129	9

NOTE: (lb/min) x (7.56) = (g/sec)
(lbs) x (2.2) = (kg)

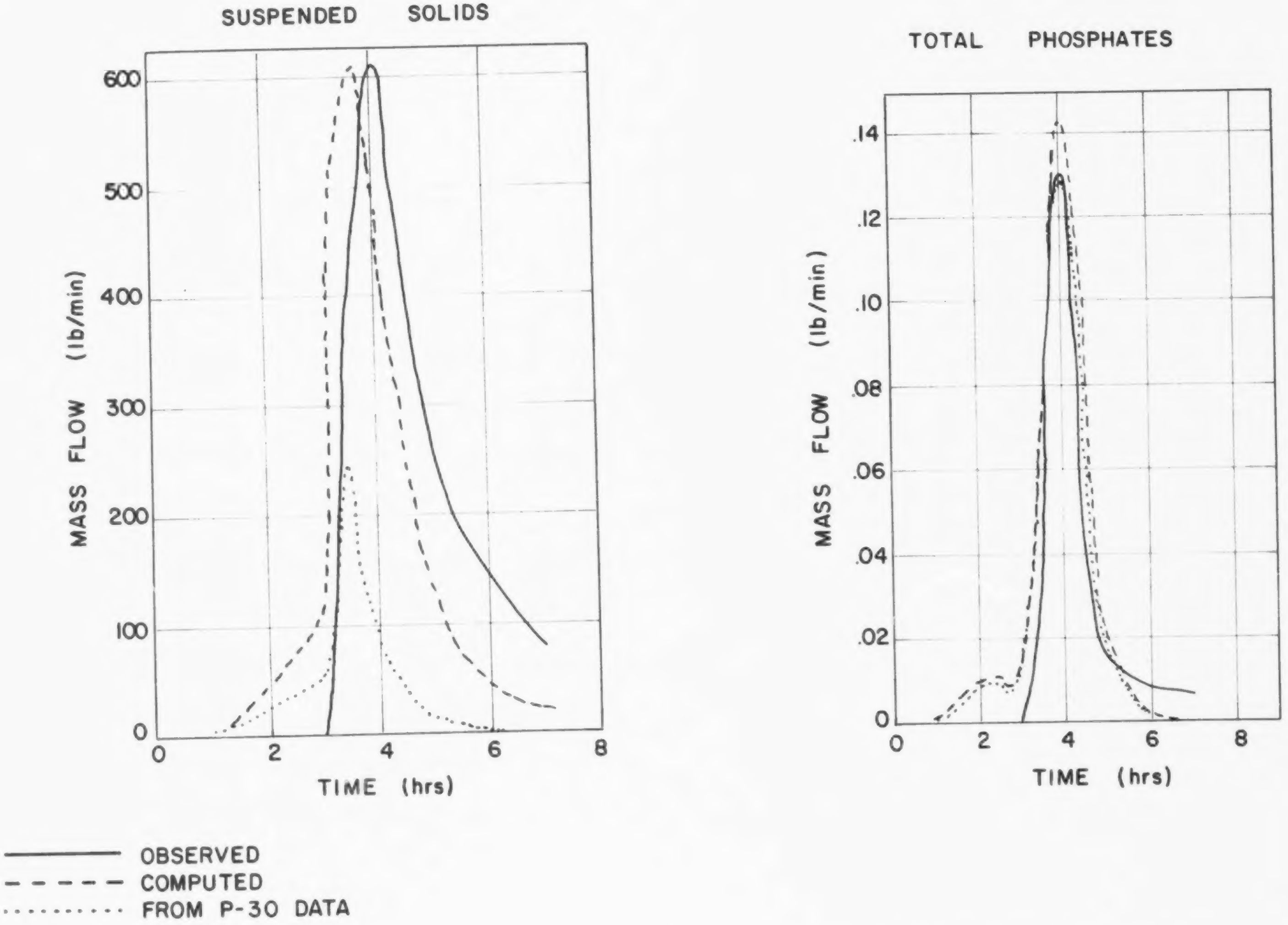


Fig. 61 Storm of 4/08/75 - Station D-10, Suspended Solids and Phosphates

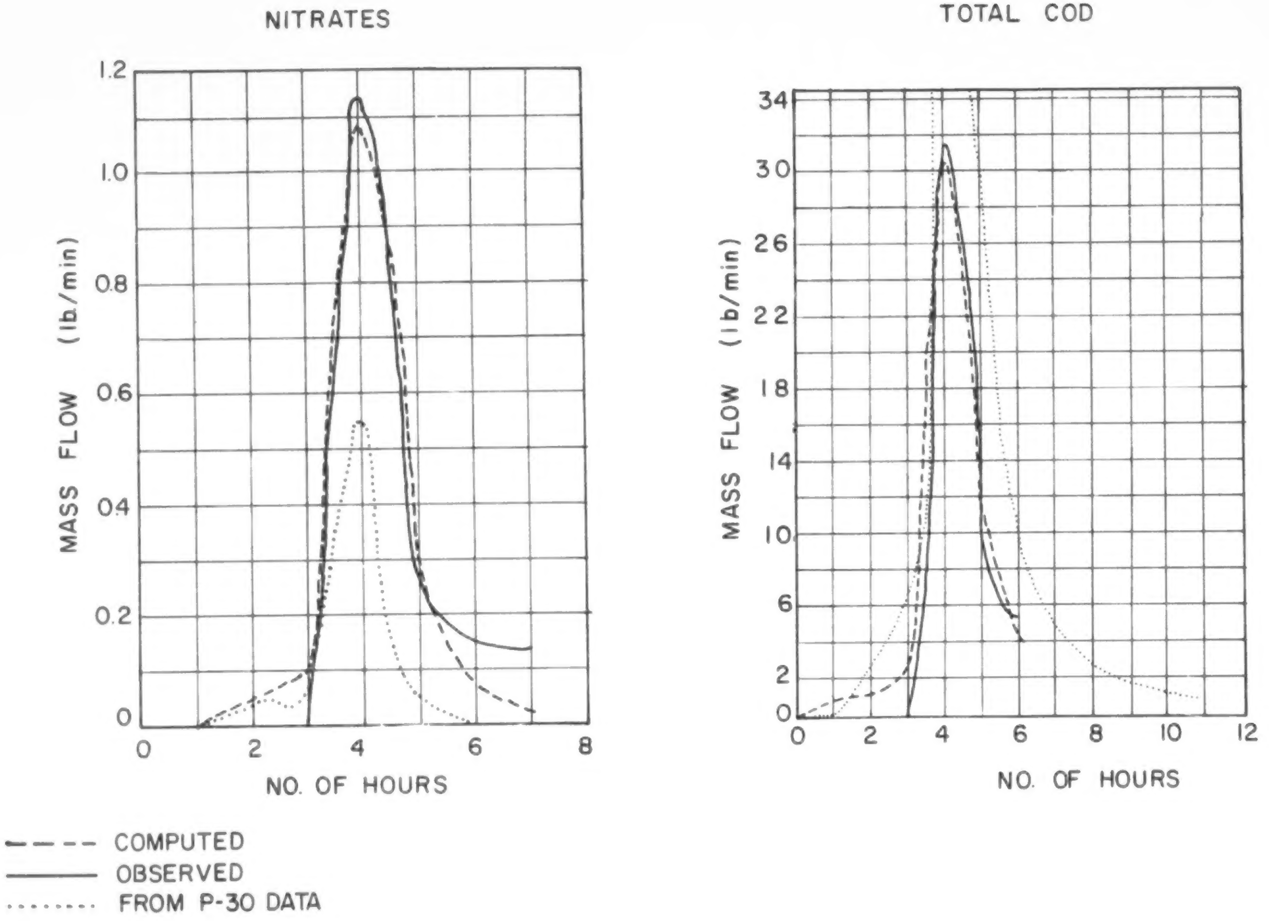


Fig. 62 Storm of 4/08/75 - Station D-10, Nitrates and Total COD

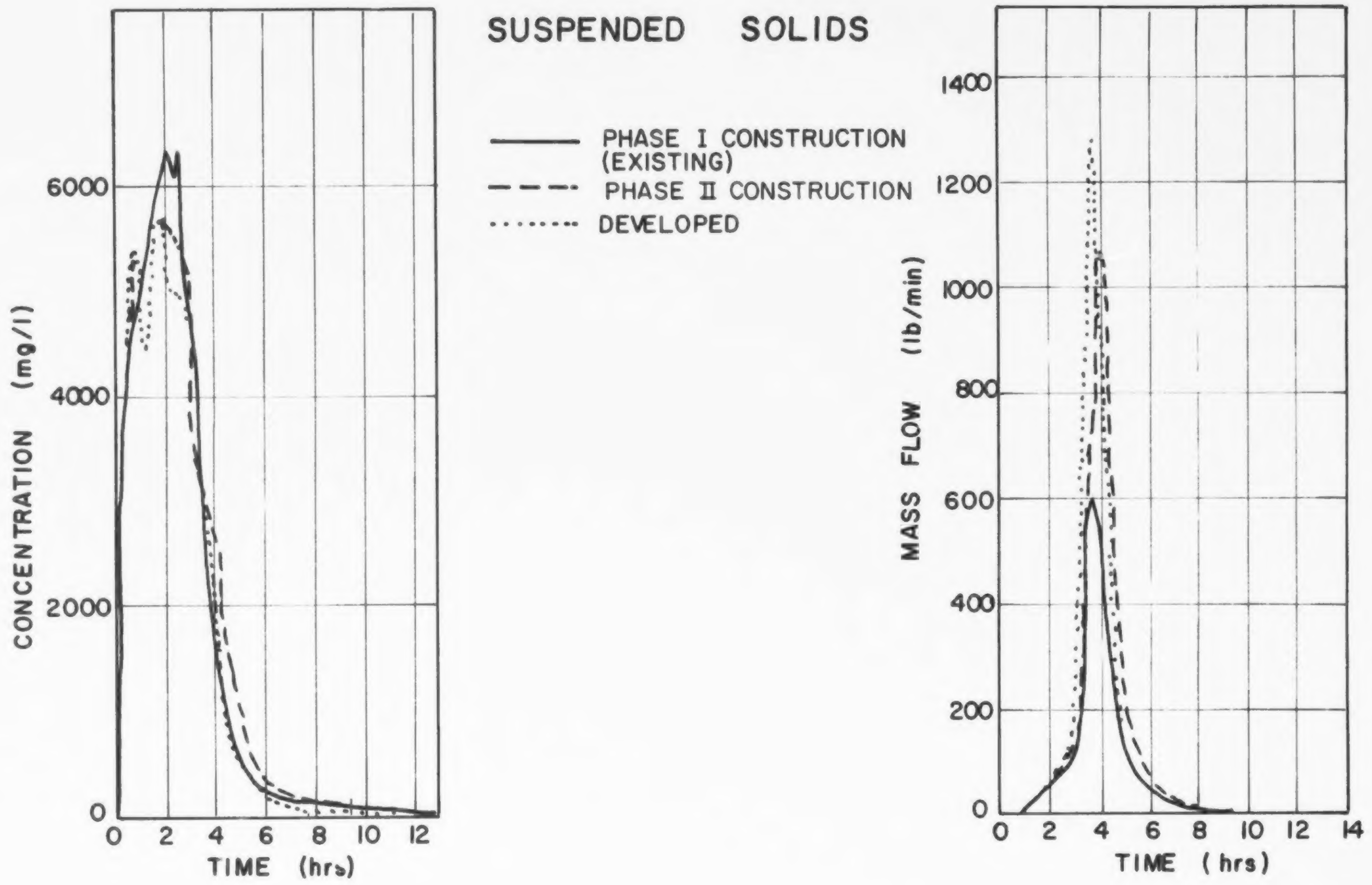


Fig. 63 Station D-10, Future Development Conditions, Suspended Solids

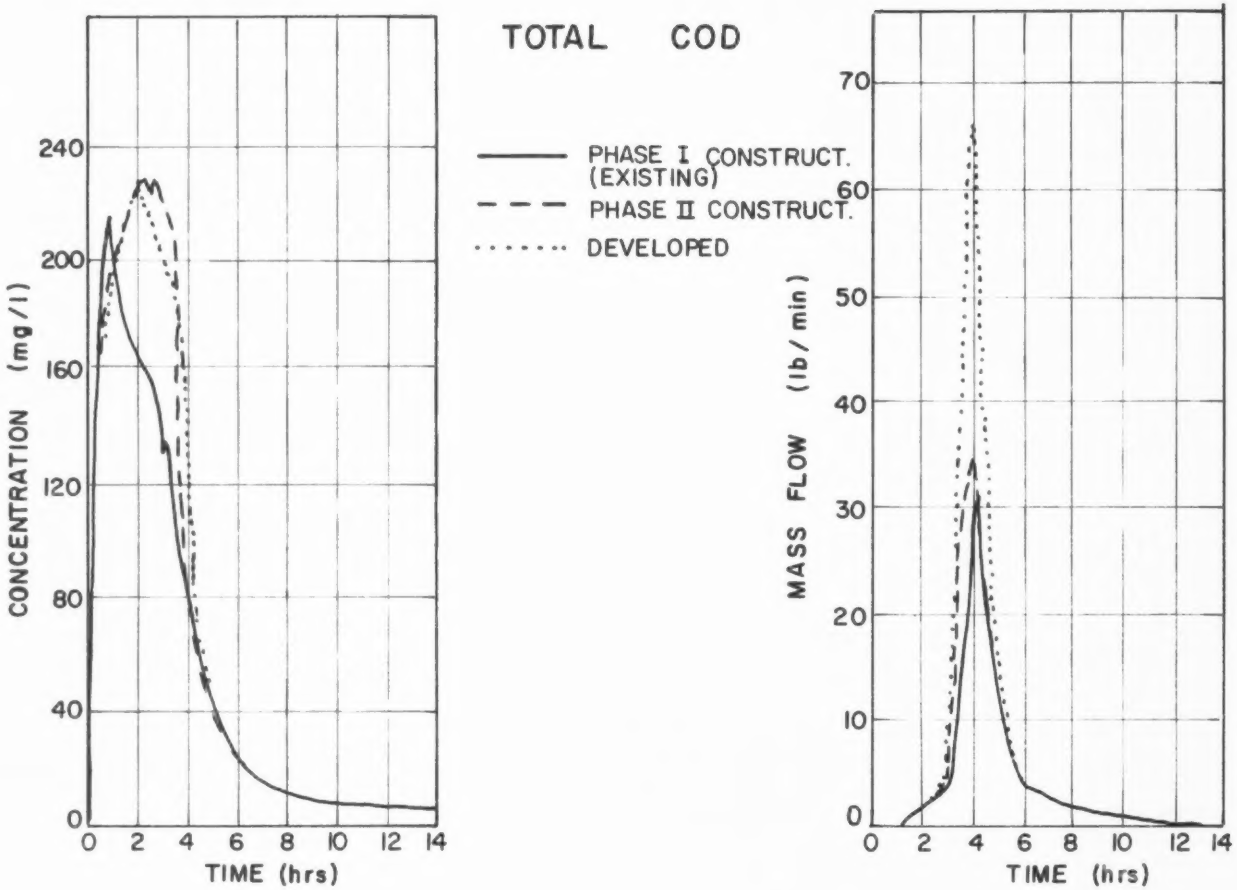


Fig. 64 Station D-10, Future Development Conditions, COD

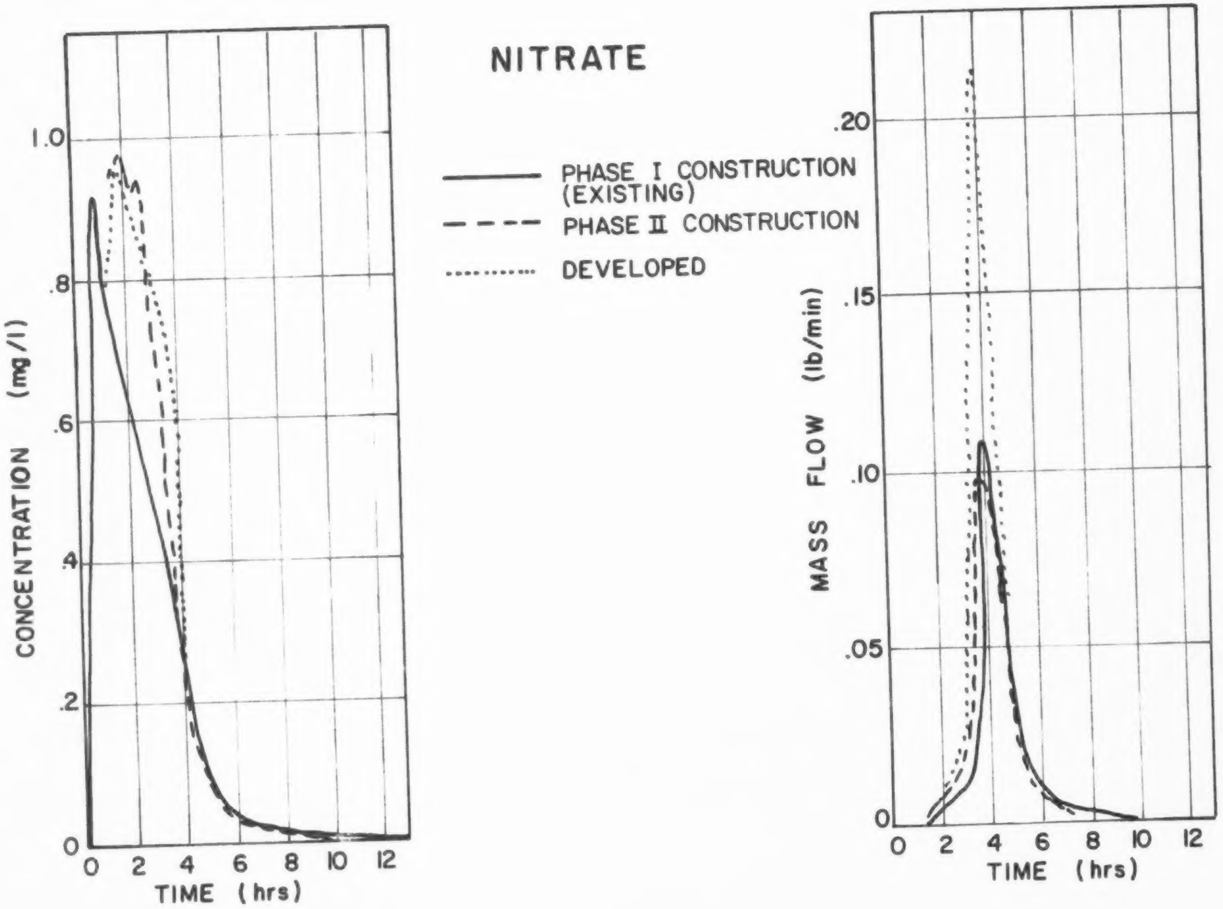


Fig. 65 Station D-10, Future Development Conditions, Nitrates

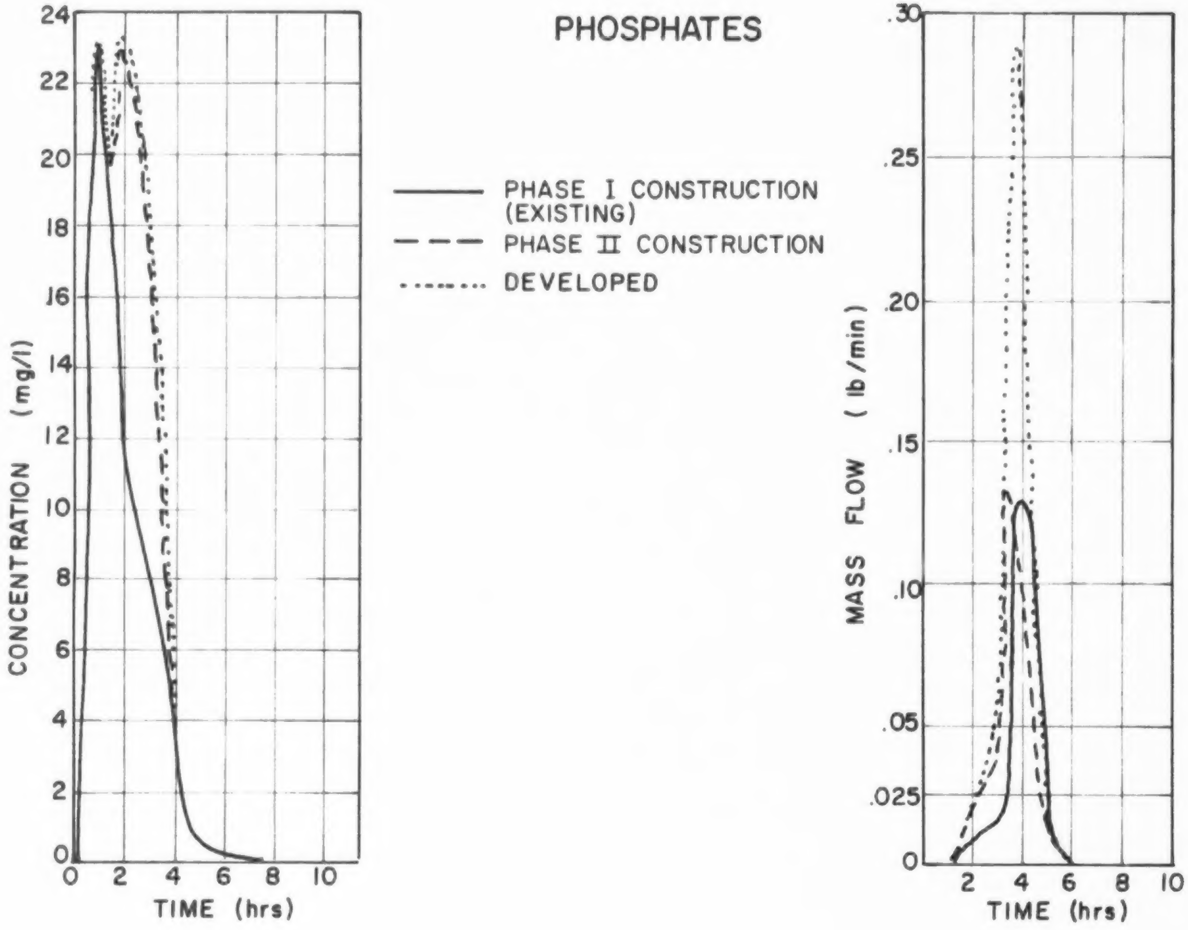


Fig. 66 Station D-10, Future Development Conditions, Phosphates

TABLE 41. MODELING RESULTS FOR FUTURE DEVELOPMENT UPSTREAM FOR STATION D-10

	Peak Conc. mg/L	Peak Mass lb/min	Total Pounds
		EXISTING CONDITIONS	
Suspended Solids	6348.	609.	65290
COD	217.	31.	3539
Nitrates	.93	.108	9
Phosphates	2.32	.129	9
		CONSTRUCTION CONDITIONS	
Suspended Solids	5713.	1080.	97163
COD	231.	35.	4232
Nitrates	.98	.099	11
Phosphates	2.34	.133	12
		DEVELOPED CONDITIONS	
Suspended Solids	5706.	1289.	95415
COD	231.	68.	5862
Nitrates	.98	.214	16
Phosphates	2.34	.289	21

One reason for the increase in peak pollutant mass transport rates is the change in the runoff hydrograph. As seen in Figure 67, the hydrograph peak is increased by approximately 40%. Another reason is the increase in the input loading rates as listed in Table 39 for developed areas, which result in a doubling of peak pollutant mass transport rates for COD, nitrates and phosphates. The 20% increase in the suspended solids pollutant mass transport rate is a result of hydrograph modification due to urbanization.

To determine the relative magnitude of pollutants at Station D-10, a further investigation of water quality in Swale 8 was also conducted. For existing conditions, equal pollutant loading rates were applied to each land use area individually and then together. This analysis provided an insight into the relative pollutant generative capacity of each land use and also the effects of flow and pollutant routing in the SWMM. Both pollutographs and pollutant mass transport rates were compared and as shown in Figure 68 and Table 42, it is evident that any pollutant is transported from the residential area at the highest concentration and unit pollutant mass transport rate. The increase in imperviousness in an urban area is a major reason for this increase because the runoff intensity is increased. Due to the effect of drainage area and routing characteristics, the results presented in Figure 68 and Table 42 apply to the Swale 8 watershed only. A similar analysis of another watershed should be performed if this type of information is desired for that watershed.

In summary, the EH&A water quality modeling version greatly improved the capabilities of the SWMM. Water quality modeling results are much more dependable and observed events can be adequately simulated. Each of the storms used to test the new SWMM version, and as described in this section, was selected to present a range of flow, water quality and land use data; thus the model was tested over a range of different conditions.

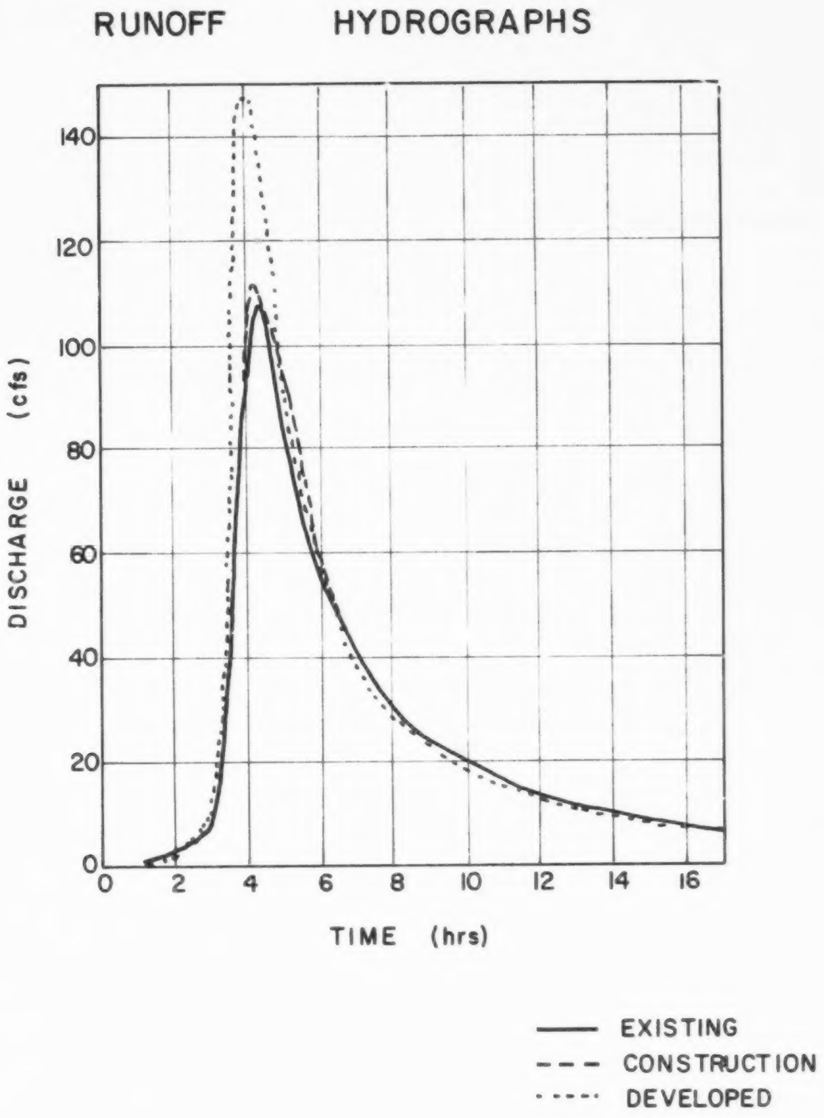


Fig. 67 Station D-10, Runoff Hydrographs - Existing and Future Conditions

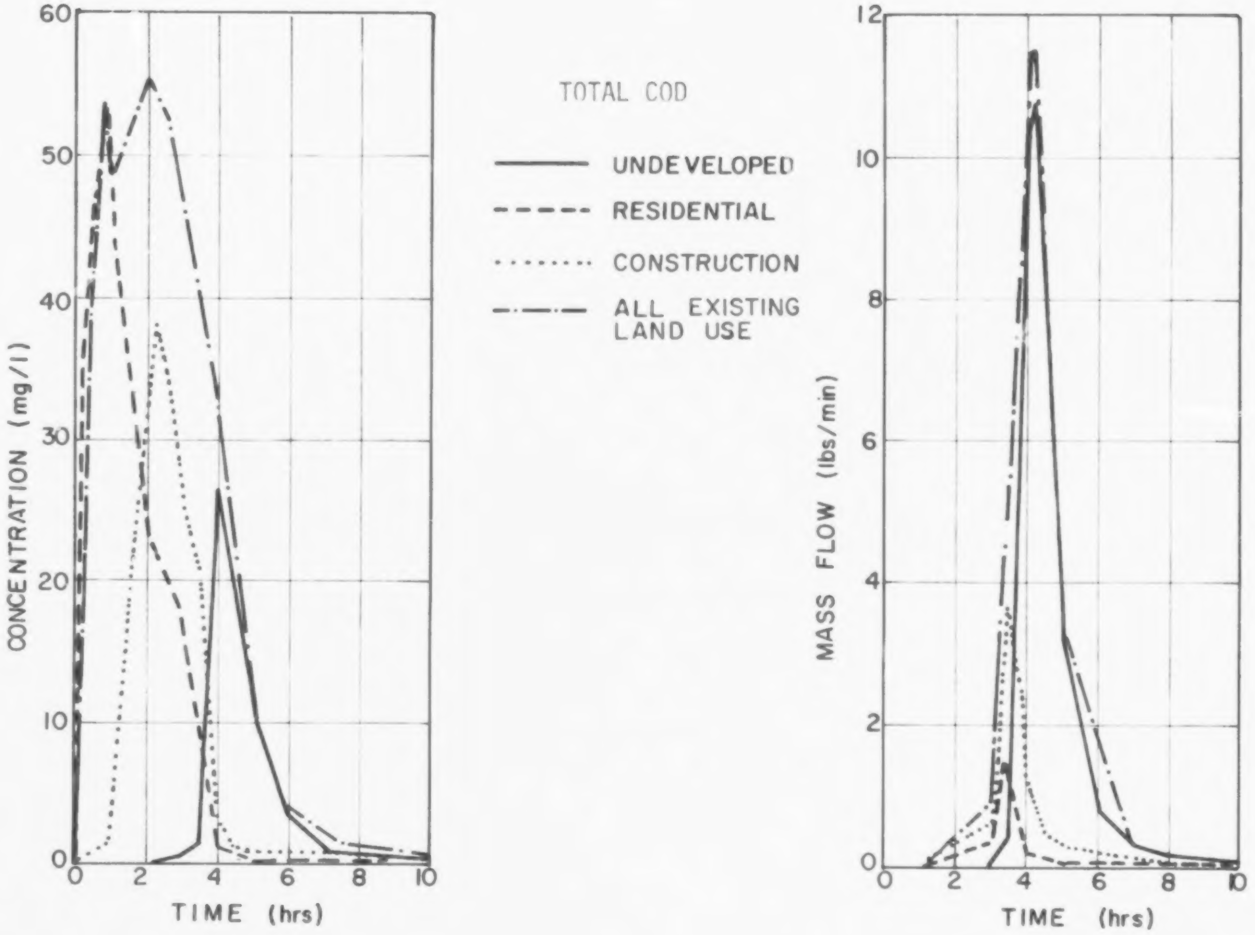


Fig. 68 Water Quality from Different Land Uses in the Swale 8 Watershed

TABLE 42. RELATIVE EFFECTS OF LAND USES IN THE SWALE 8 WATERSHED

Based on equal pollutant loading rates

Land Use	Drainage Area ac	Peak Conc. mg/L	Mass Peak lb/min	Flow Unit lb/min/ac
Natural	232	27	10.3	.043
Urban	25	54	1.4	.056
Construction	81	38	3.6	.044
All	338	56	11.6	.034

SECTION 9
SUMMARY

This study has indicated the magnitude of environmental impact by construction activity. The SWMM was used in an attempt to model this impact and the resulting effort was only partially successful. The need for further data accumulation was highlighted. The need for specific data requirements was identified and established, and a thorough review of the methodology and programming for the SWMM was conducted.

Runoff quantity and quality from areas not served by combined sewers can now be modeled by SWMM with a significant reduction in input data requirements. The SWMM can now be used to determine baseflow recessions, as well as to quantify the effects of groundwater depletion by urbanization. Generally the increase in baseflow recession rates is directly proportional to groundwater storage depletion by urbanization or other activity by man. The SWMM can also be used to design and evaluate natural drainage systems, and again the input data is significantly reduced. Only a coordinate cross section description and the channel characteristics are required.

A further development of the SWMM also allows for an evaluation of the relative efficiency of economies between natural and conventional drainage systems.

The porous pavement system model can assist in the planning of urban development by determining the hydrologic response to a design storm. The runoff rate and flow volume reduction can be evaluated. Also, to quantify the effect of pavement clogging by extraneous sources, the permeability and porosity of the pavement or base may be reduced accordingly. The status of flow and storage in the porous pavement system at all times, as shown in Table 3, will indicate the efficiency of operation as well as provide guidance in sizing the pavement and base thickness and areal extent.

The model output can facilitate decisions to be made regarding stormwater quality control. Porous pavement systems should be designed to retain, as a minimum, the intial 30 percent of runoff. Physical, chemical or biological treatment may be utilized within the system or the stored storm water can be pumped or drained to a treatment plant when treatment capacity becomes available. The possibility of evaluating dilution effects on pollutant concentrations in the stored storm water needs to be investigated.

A novel approach to water quality prediction has been developed. As more data becomes available, the predictive capabilities of this method can be refined and specific pollutional characteristics can be accentuated. The limited data at The Woodlands and at Hunting Bayou indicate the strong potential for this type of approach.

If the water quality relationships derived in this study are comprehensively developed, and the data used are sufficient to cover all types of land uses as well as geographical areas, then the universalization of site specific data will be simplified and the applicability of the SWMM will have been tremendously improved. This remains as the best avenue for further research because of the dual functions it serves - to improve definition of nonpoint source water quality characteristics as well as to indicate the transferability of data. It is suggested that new methods of water quality prediction, such as the one developed during this project, be given immediate attention.

The intensive construction activity throughout The Woodlands for the duration of this project provided an insight to the extreme pollutional loading, especially in suspended solids, that a receiving water body experiences when soil is moved and drainage systems are altered even though the changes are minimal. The desirability of porous pavement usage and natural drainage system implementation has been established and the hydrologic characteristics of these innovations have been sufficiently quantified to allow their detailed design.

Water quantity and quality from undeveloped areas (Station P-10) were modeled very well by use of the SWMM. The relatively low pollutant concentrations were well duplicated for most observed events even though the curb length for pollutant generation is a calibrated parameter. The new natural section routine facilitated input data preparation for the Transport Block.

The marked increase in suspended solids in the runoff from developing areas proved to be difficult to model. The coefficients of the Universal Soil Loss Equation have to be calibrated to model erosion; and more importantly, erosion from agricultural areas is different when compared to erosion from construction areas. Consequently, the SWMM could not be calibrated to model erosion. The relative sucess on modeling the magnitude of erosion by the modified method is inherent in the input data for this method. The user selected loading rates are necessarily high.

The most severely polluted runoff comes from urbanized areas (Station H-20). The modified (EH&A) method was more accurate than original SWMM to generate the high concentrations in this case.

The lakes constructed in the Swale 8 watershed have performed well their design function to retard excess runoff from urbanized areas and to act as clarifiers for suspended sediment removal. Although the lack of comprehensive data prevented a detailed analysis of Lakes A and B, the inflow and outflow hydrographs and pollutographs show how truly effective the lakes can be. It is believed that Lake C behaves similarly, thereby compounding the modeling problem but alleviating the problem of receiving water pollution from construction activity.

Future development in Swale 8 watershed is not expected to significantly increase the pollution transport rate from the watershed. Of course the assumption is made that only natural drainage systems will be constructed and the development plans will not be altered.

The SWMM has undergone extensive evaluation and modification. It has proved to be applicable in most areas; the only limitations being areas with a transient land use and other areas where extremely high suspended solids concentrations are generated. The modifications to the SWMM have improved its capabilities considerably. The model can be applied universally but the modeling results are highly dependent on the availability of local data.

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